Final report

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Waste Management
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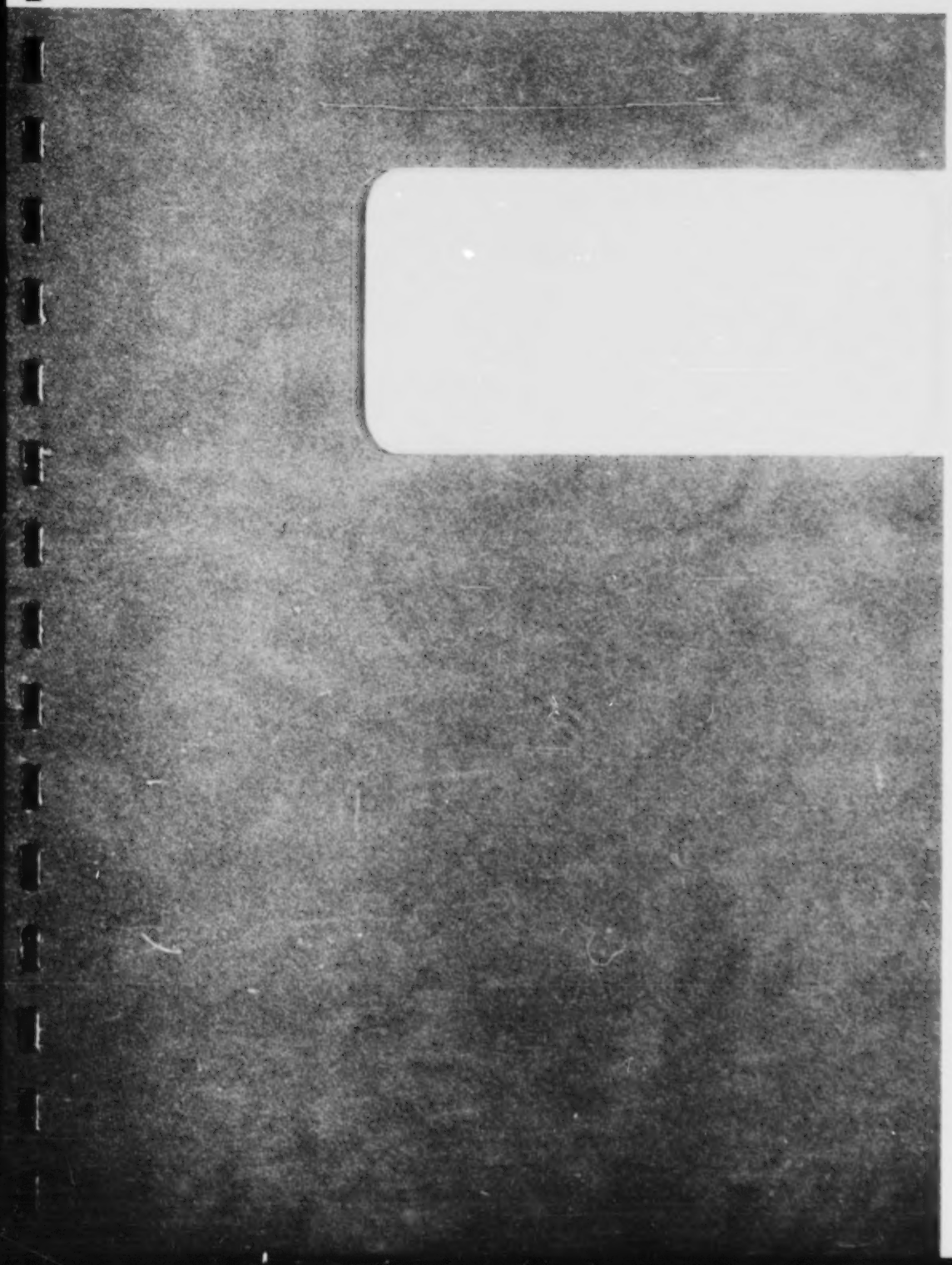

Gartner
Lee
Associates
Limited

SITE ASSESSMENT
PHASE 4B:
GEOLOGY, HYDROGEOLOGY, AND GEOTECHNICS - GEOTECHNICAL INPUT TO FACILITIES DESIGN

Prepared for
The Ontario Waste Management Corporation

Submitted By

Gartner Lee Limited

October, 1987

Gartner Lee Limited

140 Renfrew Drive,
Markham, Ontario
L3R 6B3
Telex 06-986278
Fax (416) 477-1456

(416) 477-8400

Professional Services in Environmental Management

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October 30, 1987 GLL 87562

Mr. J. G. Micak
Manager, Environmental Projects
Ontario Waste Management Corporation
2 Bloor Street West
11th Floor
Toronto, Ontario
M4W 3E2

Attention: Mr. D. F. Chollak
Environmental Planner

Dear Sirs:

Re: Site Assessment Phase 4B: Geology, Hydrogeology and Geotechnics - Geotechnical Input to Facilities Design

We are pleased to provide you with our above noted report. This report is one of six Phase 4B companion reports describing baseline conditions, hydrogeologic inputs to landfill design, geotechnical inputs to facilities design, potential impacts of the landfill on ground water, potential impacts of the central operating area on ground water and ground water management strategies.

We thank you for this opportunity to be of service. Should you have any questions please do not hesitate to call.

Yours very truly,

GARTNER LEE LIMITED

E. G. Anderson, P.Eng.
Consulting Hydrogeologist
Project Director

R. D. Powell, P.Eng.
Project Engineering
Geologist

GWR:ks

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1.0 INTRODUCTION

1.1 BACKGROUND

This report documents a portion of the geoscience studies (which incorporates geology, hydrogeology and geotechnics) carried out as part of Phase 4B of the Ontario Waste Management Corporation's (OWMC) Facilities Development Process. Phase 4B involves detailed assessments of the Preferred Site and surrounding area based on several disciplines, including the geosciences. The Preferred Site and immediate vicinity are illustrated in Figure 1-1.

The specific objectives of Phase 4B are to:

1. Define the existing environmental conditions (baseline) of the Preferred Site,

2. Predict and assess potential risks and impacts that may be associated with the proposed facility design and operations,

3. Identify and develop appropriate mitigative measures to minimize the potential risks and impacts,

4. Interpret residual risks and impacts after mitigation,

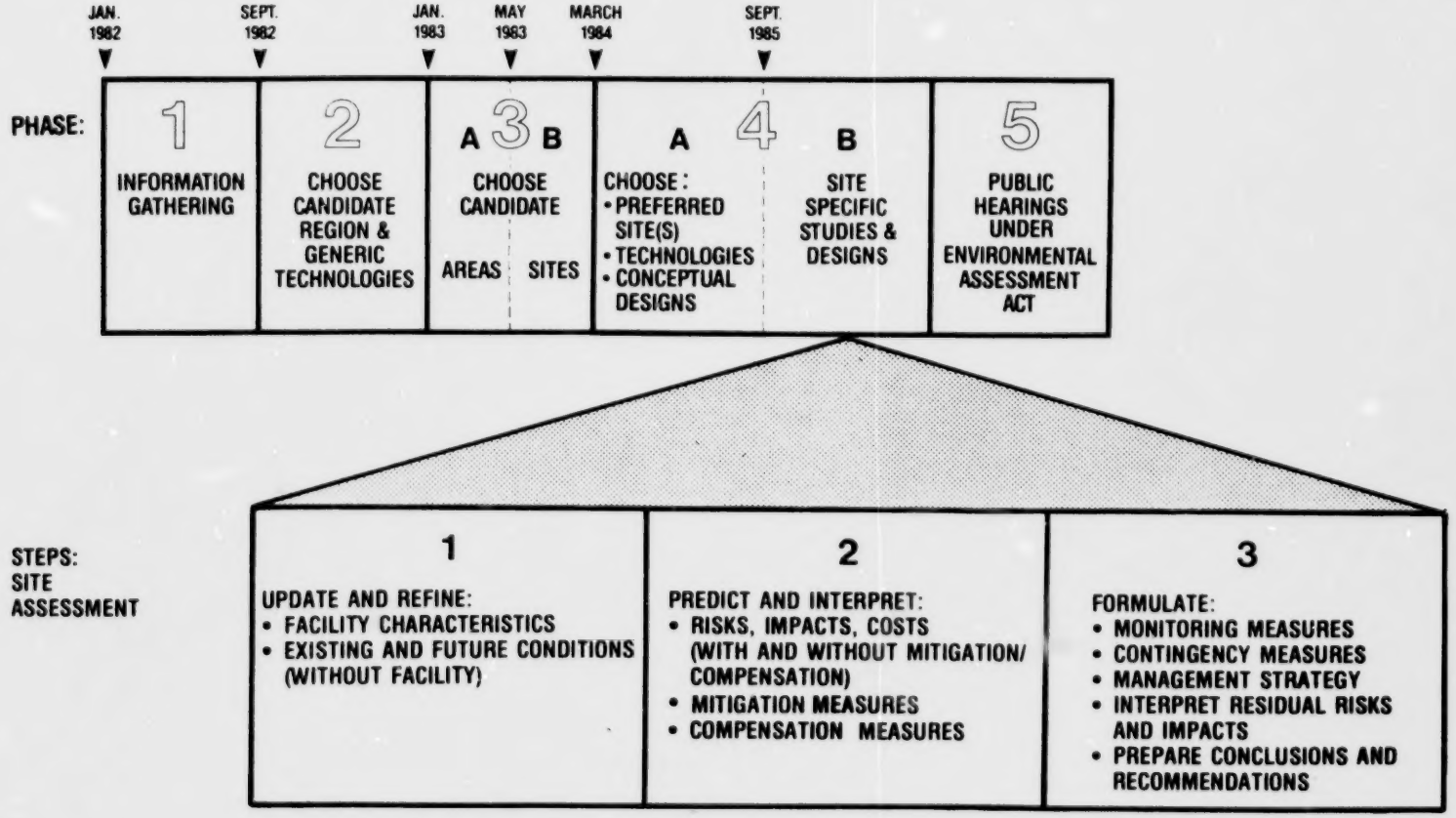
5. Identify and develop monitoring programs that will be implemented to assess the effectiveness of the design and operations in minimizing risks and impacts,

6. Identify and develop contingencies that will be implemented to minimize risks and impacts, and

7. Prepare documentation to meet the requirements of Ontario's Environmental Assessment Act.

Figure 1-2 illustrates how Phase 4B relates to the other phases of the OWMC Site Assessment and Facilities Development Process.

Township of West Lincoln, Regional Municipality of Niagara. Local Detail.



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FACILITIES DEVELOPMENT PROCESS

Figure 1.2

For the geosciences, Gartner Lee Limited's (GLL) Phase 4B studies have focussed on:

1. Establishing baseline conditions for the Preferred Site (GLL 1987a),

2. Providing input to OWMC's engineering consultants for conceptual facilities design (this report and GLL 1987b),

3. Assessing potential ground water impacts from the Central Operating Area (GLL 1987d),

4. Assessing, mitigating and comparing various engineered landfill design concepts as input to OWMC's selection of the shallow entombed landfill (GLL 1987b)

5. Assessing potential ground water impacts from the shallow entombed landfill, (GLL 1987c), and

6. Developing an overall ground water management plan through monitoring, decommissioning and remedial strategies (GLL 1987e).

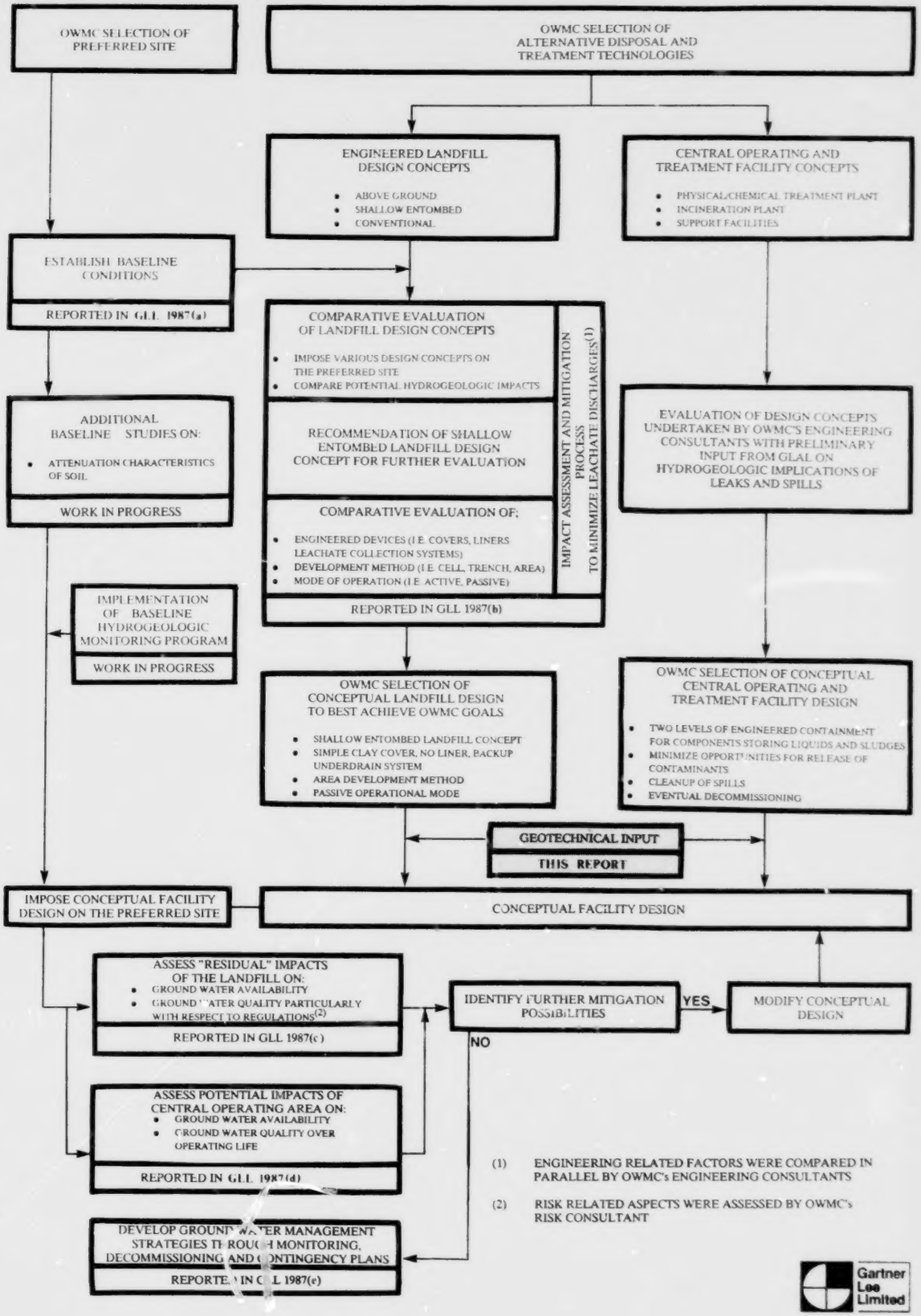
1.2 PURPOSE

This report is a compilation of the geotechnical information previously provided to OWMC's engineering consultants during Phase 4B, as input to the conceptual design of the proposed facilities at OWMC's Preferred Site. This information was initially provided to the engineering consultants through letters, memos, meetings and/or telephone conversations. The contents of this report supersede all previous correspondence. Figure 1-3 illustrates how this report fits into GLL's Phase 4B studies.

The principal objectives of the geotechnical studies undertaken during the Phase 4B geoscience program were to provide:

- a geotechnical classification of the subsoils and a preliminary assessment of the engineering properties of the subsoils at the Preferred Site;

FIGURE 1-3
PHASE 4B: IMPACT ASSESSMENT APPROACH: Geoscience


OWMC SELECTION OF PREFERRED SITE
OWMC SELECTION OF ALTERNATIVE DISPOSAL AND TREATMENT TECHNOLOGIES
ENGINEERED LANDFILL DESIGN CONCEPTS
ABOVE GROUND
SHALLOW ENTOMBED
CONVENTIONAL
CENTRAL OPERATING AND TREATMENT FACILITY CONCEPTS
PHYSICAL/CHEMICAL TREATMENT PLANT
INCINERATION PLANT
SUPPORT FACILITIES
ESTABLISH BASELINE CONDITIONS
REPORTED IN GLL 1987(a)
COMPARATIVE EVALUATION OF LANDFILL DESIGN CONCEPTS
IMPOSE VARIOUS DESIGN CONCEPTS ON THE PREFERRED SITE
COMPARE POTENTIAL HYDROGEOLOGIC IMPACTS
RECOMMENDATION OF SHALLOW ENTOMBED LANDFILL DESIGN CONCEPT FOR FURTHER EVALUATION
COMPARATIVE EVALUATION OF;
ENGINEERED DEVICES (I.E. COVERS, LINERS LEACHATE COLLECTION SYSTEMS)
DEVELOPMENT METHOD (I.E. CELL, TRENCH, AREA)
MODE OF OPERATION (I.E. ACTIVE, PASSIVE)
IMPACT ASSESSMENT AND MITIGATION PROCESS TO MINIMIZE LEACHATE DISCHARGES(1)
REPORTED IN GLL 1987(b)
EVALUATION OF DESIGN CONCEPTS UNDERTAKEN BY OWMC'S ENGINEERING CONSULTANTS WITH PRELIMINARY INPUT FROM GLAL ON HYDROGEOLOGIC IMPLICATIONS OF LEAKS AND SPILLS
ADDITIONAL BASELINE STUDIES ON:
ATTENUATION CHARACTERISTICS OF SOIL
WORK IN PROGRESS
IMPLEMENTATION OF BASELINE HYDROGEOLOGIC MONITORING PROGRAM
WORK IN PROGRESS
OWMC SELECTION OF CONCEPTUAL LANDFILL DESIGN TO BEST ACHIEVE OWMC GOALS
SHALLOW ENTOMBED LANDFILL CONCEPT
SIMPLE CLAY COVER, NO LINER, BACKUP UNDERDRAIN SYSTEM
AREA DEVELOPMENT METHOD
PASSIVE OPERATIONAL MODE
OWMC SELECTION OF CONCEPTUAL CENTRAL OPERATING AND TREATMENT FACILITY DESIGN
TWO LEVELS OF ENGINEERED CONTAINMENT FOR COMPONENTS STORING LIQUIDS AND SLUDGES
MINIMIZE OPPORTUNITIES FOR RELEASE OF CONTAMINANTS
CLEANUP OF SPILLS
EVENTUAL DECOMMISSIONING
GEOTECHNICAL INPUT
THIS REPORT
IMPOSE CONCEPTUAL FACILITY DESIGN ON THE PREFERRED SITE
CONCEPTUAL FACILITY DESIGN
ASSESS "RESIDUAL" IMPACTS OF THE LANDFILL ON:
GROUND WATER AVAILABILITY
GROUND WATER QUALITY PARTICULARLY WITH RESPECT TO REGULATIONS(2)
REPORTED IN GLL 1987(c)
IDENTIFY FURTHER MITIGATION POSSIBILITIES
YES
MODIFY CONCEPTUAL DESIGN
NO
ASSESS POTENTIAL IMPACTS OF CENTRAL OPERATING AREA ON:
GROUND WATER AVAILABILITY
GROUND WATER QUALITY OVER OPERATING LIFE
REPORTED IN GLL 1987(d)
DEVELOP GROUND WATER MANAGEMENT STRATEGIES THROUGH MONITORING, DECOMMISSIONING AND CONTINGENCY PLANS
REPORTED IN GLL 1987(e)
(1) ENGINEERING RELATED FACTORS WERE COMPARED IN PARALLEL BY OWMC's ENGINEERING CONSULTANTS
(2) RISK RELATED ASPECTS WERE ASSESSED BY OWMC's RISK CONSULTANT
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- an adequate geotechnical data base to assist in the conceptual design of the engineered landfill proposed for the Preferred Site; and

- geotechnical input into the conceptual design of the facilities and general construction considerations.

The data presented in this report are still preliminary in nature and are only intended for use in conceptual design. Detailed geotechnical investigations will be required prior to final engineering design of the landfill and related structural facilities.

This assessment of the engineering properties of the subsoils and all supporting geotechnical data are contained in the companion document entitled Site Assessment - Phase 4B: Geology, Hydrogeology and Geotechnics, Baseline Conditions (GLL, 1987a). An engineering interpretation of the geotechnical characteristics of the subsoils, as they may affect the conceptual design of the facilities is presented in this report. This report should be read in conjunction with the GLL, 1987a report.

1.3 SCOPE AND METHODS

Geotechnical studies were undertaken as part of Phase 4A and Phase 4B to provide OWMC and its engineering consultants (Monenco and M.M. Dillon) with a preliminary assessment of the engineering properties and behaviour of the the soils at the Preferred Site.

The Phase 4A geotechnical interpretation was based mainly on a review of previously published geotechnical studies in the general geographic area, supplemented by limited geotechnical data obtained from five preliminary assessment boreholes completed on-site in 1984. The major conclusions of the 4A report were that the landfill and site facility could be engineered from a geotechnical standpoint and that detailed site specific studies should be carried out to provide geotechnical design information suitable for preliminary, then final design.

The Phase 4A geotechnical information was used by the engineering consultants as input to the feasibility design of foundations and structures for the comparative

evaluation of siting options (Phase 4A Site Selection Process). This information was also used in the development of the engineered landfill concepts on-site.

The Phase 4B geotechnical assessment of subsurface conditions at the Preferred Site builds on the Phase 4A geotechnical interpretation. The work plan was designed to provide the engineering consultants with additional information on:

- soil slope stability for the engineered landfill and other excavations;
- preliminary foundation considerations; and
- operational considerations for the clayey soils with respect to excavation methods and trafficability for vehicles and equipment.

Following the selection by OWMC of the shallow entombed landfill concept (Monenco, 1987), the scope of the geotechnical assessment was modified to accommodate the design requirements of this preferred concept.

This report presents the interpretation of the geotechnical findings as they relate to stability of excavations, handling of the soil and other construction considerations.

Published information, including the Phase 4A report, Welland Canal studies, and other data in the general geographic area were compiled and reviewed. These data provided insight into the general subsurface conditions, as well as an indication of the physical properties of the soils. This information is summarized in the GLL, 1987a report and the pertinent references are listed in Appendix C of this report.

The field investigation consisted of two main components:

(1) subsurface drilling, soil and rock sampling and monitoring of ground water installations; and

(2) a review of natural slopes along the Welland River.

A detailed description of the drilling and sampling protocols is presented within Appendix A1 of the GLL, 1987a report. Information from the Phase 4A and 4B boreholes which formed the basis for the evaluation of the geotechnical conditions at the Preferred Site are in Appendix C1 (GLL, 1987a).

A brief reconnaissance survey of the natural slopes along portions of the Welland River, directly south of the site, was conducted during June, 1986. The objective of this work was to observe the long term behaviour of natural slopes developed in similar clay-type soils. The results of this survey are discussed in Section 3.2.1 of this report.

Laboratory testing was also carried out on a wide selection of soil samples to determine, a) the index properties, b) the drained and undrained shear strengths and c) the consolidation characteristics of the various soils units. All of the laboratory testing was carried out by Golder Associates (Eastern Canada) Ltd. who made the actual selection of the test specimens from each specified shelby tube or split spoon sample. Results of the Golder testing are presented in Appendix E1 of the GLL, 1987a report.

Results of the laboratory program, together with the results of the drilling program formed the basis of the preliminary geotechnical assessment of the Preferred Site. The compilation of data and a discussion of the soil units from a geotechnical standpoint are presented in Section 6.0 of the GLL, 1987a report.

Comments respecting materials handling, stockpiling, the placement of cover materials and for other construction considerations, are based largely on the preliminary soils engineering properties and documented observations of construction activities in the general geographic area of the Preferred Site.

This report provided direct factual input to the engineers for the conceptual design of the landfill and facilities. No factors were used for the preparation or interpretation of this report.

1.4 ASSUMPTIONS AND BACKGROUND DATA

1.4.1 Baseline Conditions

The baseline geologic, hydrogeologic and geotechnical conditions at the Preferred Site are described in detail in the report entitled "Site Assessment Phase 4B: Geology, Hydrogeology and Geotechnics - Baseline Conditions" (GLL 1987a). The site characteristics detailed in that report are the base upon which the landfill design has been imposed in this report to predict the potential impacts. An overview of the baseline geology and subsurface conditions is provided in Chapter 2 of this report.

1.4.2 Landfill Design & Facilities

The conceptual design for the shallow entombed landfill at the Preferred Site is described in the Monenco, 1987 and has been assumed for the impact analyses presented in this report. A brief description of the landfill design and operation is given in Chapter 3.

As previously discussed, this report is a compilation of data requested by the Engineers as input to the design of the facilities. No direct input by the Engineers is presented in this report as this is a factual discussion of conceptual designs.

1.4.3 Status of Data

As more subsurface investigations are carried out for the design of the facilities, additional geotechnical data will become available. These data will provide a broader base on which to design and estimate performances.

1.5 REPORT FORMAT

The remainder of this report is organized as follows:

Chapter 2 - overview of baseline conditions

Chapter 3 - assessment of geotechnical conditions as they relate to the design of the landfill

Chapter 4 - assessment of geotechnical conditions as they relate to the design of the facility.

2.0 BASELINE CONDITIONS

2.1 SUMMARY OF SOIL ENGINEERING PROPERTIES

The following description of the engineering properties of the overburden materials (soils) underlying the Preferred Site is derived from Section 6.0 of the GLL, 1987a report.

The geologic names of the four main stratigraphic units are retained for use in this report. These units, proceeding vertically from ground surface to depth (i.e., youngest to oldest) are:

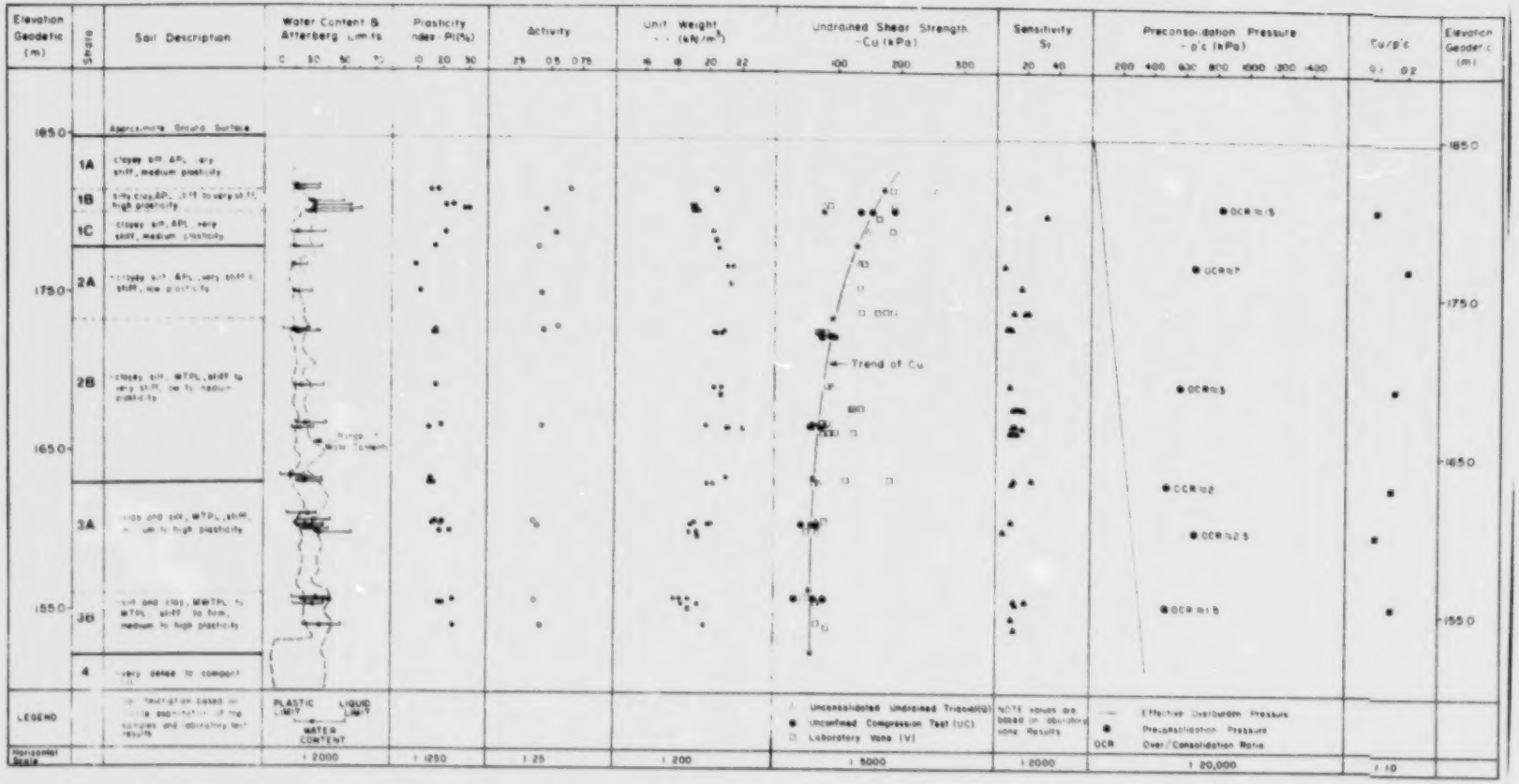
(1) Upper Glaciolacustrine Unit;
(2) Halton Unit;
(3) Lower Glaciolacustrine Unit; and
(4) Lower Till.

The results of the laboratory testing program are summarized in Figure 2-1. The results illustrate that there are differences in the soil properties both between, and within, the main stratigraphic units mentioned above. These differences within each stratigraphic unit allowed the units to be subdivided into smaller subunits. The soil properties within each subunit are similar across the site. A brief summary of the changes in physical properties between the units and subunits is presented in Table 1. A more detailed discussion of the soil engineering properties of the various units and subunits is presented in Section 6.0 of the GLL, 1987a report.

In general terms, the stratigraphy and summary plots shown in Figure 2-1 display the following trends:

- Six consolidation test data points suggest a marked increase in the preconsolidation pressure towards surface. This corresponds with a general increase in shear strength toward surface and corresponding water contents that are near the plastic limit near surface.

FIGURE 2-1 VARIATION OF PHYSICAL SOIL PROPERTIES WITH ELEVATION (SUMMARY PLOT)



Legend

1 Upper Glaciolacustrine Unit

2 Halton Unit

3 Lower Glaciolacustrine Unit

4 Lower Till Unit

A-C Indicates Subunits

DTPL - Drier than plastic limit

APL - About the plastic limit

WTPL - Wetter than plastic limit

MWTPL - Much wetter than plastic limit

+ - Values used for stability analyses

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TABLE 1 VARIATION OF INDEX PROPERTIES BETWEEN SUBUNITS

UNIT	SUB-UNIT	SOIL CHARACTERISTIC PROPERTIES	BASIS FOR DIVISION
1-Upper Glacio-lacustrine Unit	1A	clayey silt, very stiff medium plasticity	- increased plasticity and water content within 1B - lower unit weight and shear strength within 1B
	1B	silty clay, stiff to very stiff, high plasticity	- within 1C the plasticity decreases and the strength and unit weight increases.
	1C	clayey silt, very stiff, medium plasticity	- the plasticity of subunit 2A is lower than 1C and the unit weight is higher. However, the undrained strength of the two subunits (2A and 1C) is similar.
2-Halton Unit	2A	clayey silt, very stiff to stiff, low plasticity	- the main differences between subunits 2A & 2B are the increased moisture content and slightly decreased strength in 2B.
	2B	clayey silt, very stiff to stiff, low to medium plasticity	- the contact between 2B & 3A is highlighted by an increase in water content and decrease in unit weight. This is a reflection of the increased clay content within the Lower Glaciolacustrine Unit.
3-Lower Glacio-lacustrine Unit	3A	clay and silt, stiff, medium to high plasticity	- the contact within Unit 3 has been identified based on the increased moisture content of 3B.
	3B	silt and clay, stiff to firm, medium to high plasticity	-the contact between the Lower Glacio-lacustrine Unit and Lower Till was very evident based on visual and tactile examination of the samples.
4-Lower Till	4	very dense to compact till	

- The activity is relatively constant at 0.5 to 0.4 with depth, indicating generally inactive clayey soils.

- The general decrease of undrained strength with depth extends approximately to elevation of 165 m a.s.l. This corresponds with the decrease in preconsolidation pressure with depth. Below 165 m elevation, the strength appears relatively constant, for the methods of sampling and testing employed.

- Zones of higher water content typically correlate with lower unit weights.

The elevations of the contacts between four main stratigraphic units (suggested by changes in the physical properties) were similar to the average contact elevations determined during geologic logging of the boreholes. These contact elevations were used in the stability analyses of the various slopes for the proposed landfill excavation. Based on the very dense nature of the Lower Till as determined by Standard Penetration Resistance "N" values and its stratigraphic position, the top surface of the Lower Till is considered to be the base of all potential slope stability failure planes.

Shear strength testing was carried out on soil samples from each of the subunits in order to obtain a preliminary indication of both the effective shear strength parameters (peak and residual) and the undrained shear strengths. Summaries of these results are presented in Tables 2 and 3, respectively.

TABLE 2 SUMMARY OF EFFECTIVE SHEAR STRENGTH TESTING

Sub-Unit	EFFECTIVE SHEAR STRENGTH PARAMETERS				Test Number and Type
	phi (degrees)		cohesion (kPa)		
	peak	residual	peak	residual	
1A	32	N/A	20	N/A	1-T
1B	19	N/A	0	N/A	1-T
1C	21	20	8	0	1-DS
2A	28	N/A	20	N/A	1-T
2B	22	21	20	0	1-DS
2B	29	28	7	0	1-DS
3A	26	N/A	10	N/A	1-T
3A	20	19	0	0	1-DS
3B	18.5	17	0	0	1-DS

N/A = Not Available

T = Consolidated Undrained Triaxial with pore pressure measurements

DS = Drained Direct Shear

TABLE 3 SUMMARY OF UNDRAINED SHEAR STRENGTH TESTING

Subunit	Range of Undrained Shear Strengths (1) (kPa)	Number of Tests
1A	186 to 252	2
1B	80 to 189	5
1C	146 to >186	2
2A	133 to >186	4
2B	56 to 122	14
3A	51 to 78	5
3B	33 to 83	7

NOTE:

(1) Vertically oriented tube samples and vertically oriented laboratory tests including laboratory vane, unconfined compression and undrained triaxial tests.

(2) > means greater than.

3.0 INPUT TO CONCEPTUAL DESIGN OF LANDFILL

3.1 INTRODUCTION

The shallow entombed landfill concept has been selected by OWMC as the preferred landfill concept for preliminary engineering design.

This report describes the methods and results of those analyses considered pertinent to the preliminary geotechnical input for the conceptual design and operation of the landfill.

The shallow entombed concept places all of the solidified mass below the base of the brown surface soils which consist of weathered and fissured silt and clay. The GLL, 1987a report indicates that the weathered silt and clay is about 5 metres (m) thick. A cover system, up to 5 m in thickness, will be placed above the solidified mass with the final surface completed at about original ground surface level. It is understood that an underdrain system composed of a granular blanket drain with a pipe network and wet wells or a series of sumps is currently planned for the base of the excavation for leachate collection, if necessary. The current plan is not to operate the underdrain system. The depth of excavation for the shallow entombed concept, as presently envisaged, is to be about 15 metres below existing grade. This landfill concept formed the starting point for the slope stability analyses of Phase 4B.

3.2 STABILITY OF THE EXCAVATION

3.2.1 Approach

The evaluation of the stability of the landfill excavation involved:

- A brief review of natural slopes along the Welland River south of the Preferred Site was carried out to observe the long term behaviour of natural slopes composed of similar clayey soils;

- An analytical assessment of the stability of the base of the excavation against base heave due to high uplift pressures and upward ground water flow; and

- An analytical assessment to determine the stability of slopes with various profiles.

The reconnaissance survey along the Welland River showed that the majority of the natural slopes were standing generally at an inclination of 3:1 H:V (horizontal to vertical). Based on the field observations, previously published data, and assumed loadings to the slope, the initial approach was to assess the stability of relatively flat slope profiles similar to natural slopes in the area. Thus, the initial configuration was a 15 m deep excavation with side slopes of 3:1 H:V.

Basal stability was analyzed following guidelines outlined in the Canadian Foundation Engineering Manual (1985) (CFEM, 1985). The pore water pressures, as determined from the finite element modelling of the excavation, were used in this analysis, the results of which are presented in Section 3.2.2.

Slope stability calculations were performed with the assistance of the PC-SLOPE computer program which calculates the Factor of Safety of a slope using the simplified Bishop limit equilibrium method of slices. A description of the PC-SLOPE program is presented in Appendix B. The confirm the results of the computer calculations, selected slopes were reanalyzed using the same method by hand calculator. Results of the slope stability analyses are presented in Section 3.2.3.

The pore water pressures estimated from the finite element ground water modelling of the excavation were used for both of the above analyses.

Pore water pressures have a direct effect on shear strength and subsequently on the stability of clayey slopes. Changes in pore water pressure from the initial field conditions, to the short term undrained excavated condition, to the longer term drained or partly drained operating conditions must be considered. A detailed description of the existing in-situ hydrogeologic conditions at the Preferred Site and the finite element program used in the flow regime analysis are presented in the GLL, 1987a report. Ground water conditions as they may influence the stability of the excavated landfill slopes as determined by finite element modelling are discussed below.

The modelling results indicate that once a portion of the landfill has been excavated, the existing ground water flow pattern will be altered in the vicinity of the excavation. Behind the face of the slope, ground water will flow in a curvilinear path varying from vertical to horizontally inwards towards the slope faces. Beneath the base of the excavation, flow will be in an upwards direction towards the excavation base. In terms of pore water pressures as determined by the finite element model, constant pressures of 0 kPa were assumed along the ground surface and 235 kPa along the upper surface of the Lower Till.

The ground water scenario described above has been used to analyze both the stability of the slopes and potential bottom heave of the proposed landfill excavations. The pressure head diagram used for these stability analyses is shown on Figure 3-1.

Slope stability analyses were carried out to establish the sensitivity of the safety factor to variations in a wide variety of input parameters. A list of the variables and discussion of each follows:

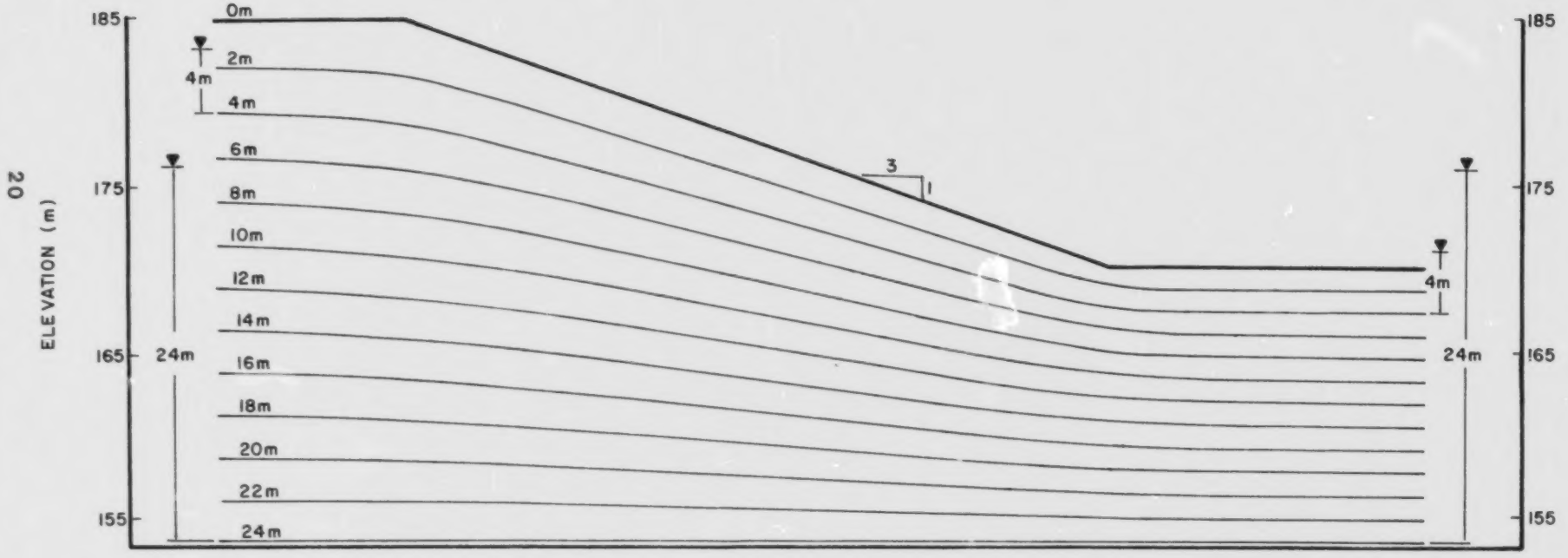
a) changes in the slope geometry including slope angle, benching, etc.

The actual slope profile scenarios selected were based on discussions with both engineering consultants Monenco and Dillon with due consideration for the geotechnical setting of relatively soft sensitive clay at depth and overconsolidated stiffer clay near surface.

Specifically, variations in the slope profile were used to compare the Factors of Safety between shallow and steeper slopes angles. Slope benching, which ultimately provides a flatter slope angle, was analyzed to assess its effect on the stability of both the lower slope (i.e. below the bench) and the entire slope overall.

b) changes in the shear strength parameters of the various soil units.

FIGURE 3-1
Pressure Head Diagram



Each "set" of soil strengths, representing one "case" or strength scenario, was used as input to a number of different slope profiles. Various "cases" representing various combinations of strengths and strength parameters were studied covering both the short and long term conditions. If an excavation is opened for a short term period in a saturated clayey soil and no volume change has taken place, the short term undrained strength of the clay is considered applicable. For long term design, values of pore water pressures from the steady state flow net are normally used along with measured long term or effective shear strength parameters.

The four specific cases analyzed are briefly discussed below:

CASE A:

This case considers the probable lower bound soil strengths for both short and long term conditions. The undrained shear strength values used in the total stress analyses (short term) were obtained by taking the minimum average shear strength value within each subunit (marked with an asterisk) off the "average" strength trend line plotted in Figure 2-1. These strengths are not the lowest measured but the lowest trend-line values on Figure 2-1. This approach to the selection of lower bound strengths assumes that individual values below the "asterisk" value represent disturbed samples.

The effective shear strength values used for long term analyses were obtained by using averages of our strength test results or previously published data where available. For Unit 1 (Upper Glaciolacustrine) a zero cohesion intercept was assumed due to the potential for the presence of fractures that would produce strength losses. Only the lower two internal angles of friction of those reported in the test data base were selected to determine an average strength of Unit 1A. The rationale for this was that the cohesion value of 32 degrees was too high for this type of soil. Strength properties for Unit 2 (Halton) were established by averaging the results of the laboratory analyses and comparing these to published data by Conlon et al., 1971. The lower test value of the two data sources was selected for use for the analysis. This same methodology was also used in establishing the strength data for Unit 3 (Lower Glaciolacustrine).

In addition, previously published data for the area (i.e. Conlon et al., 1971) also indicated design problems related to soil strength anisotropy. The above noted reasons support a conservative approach to preliminary design.

CASE B

Case B, which is considered to be more representative of the in-situ soil conditions, incorporated approximate average effective shear strength values obtained from the strength testing performed for this study. In some instances where the number of test results available for averaging was limited, these average values were modified downward.

CASES C AND D

Cases C and D were analyzed to determine the influence of increasing the effective cohesion intercept from 0 kPa (Case A) to 5 kPa (Case C) and then to 10 kPa (Case D) for both subunits 1A and 1C. The "extra" stability calculated for the upper slope using increased cohesion values may be achieved in practice due to the near surface preconsolidation. The monitoring of actual slopes along with routine ongoing testing for design and construction would provide a basis for optimizing slope configurations.

The assumed soil properties and thickness of each subunit for each of the above scenarios are summarized in Table 4.

c) the effect of additional loadings applied to the slope,

The effects of additional loadings acting on the slope were simulated by a) a 20 tonne line load along the crest of the slope; and b) the addition of a seismic coefficient 0.05 in the stability analyses. The line load actually simulated a line of trucks or other equipment on the working bench. The truck simulation may be over-emphasized in the analyses as the computer program assumes the load is of infinite length. The addition of the seismic coefficient simulates the influence of horizontal local earthquake forces as defined in CFEM, 1985 for the study area. The occurrence of an earthquake was simulated in the static stability analysis by applying an additional seismic force that acts horizontally through the centroid of each slice in the direction of potential movement.

TABLE 4: SUMMARY OF SOIL PROPERTIES USED IN STABILITY ANALYSES

CASE	UNIT*	ELEVATION OF CONTACTS (m.a.s.l.)	SOIL PROPERTIES UNIT WEIGHT (kN/m3)	COHESION (kPa)	PHI ANGLE (degrees)
A TOTAL STRESS	UGL	184.8 to 181.5	20.38	175	0
		181.5 to 180.1	19.05	80	0
		180.1 to 177.9	20.34	130	0
	HALTON	177.9 to 173.3	21.22	95	0
		173.3 to 163.0	20.60	65	0
	LGL	163.0 to 156.0	19.26	61	0
		156.0 to 152.1	18.62	61	0
	LOWER TILL	152.1 TO 149.3	BASE OF ANALYSIS		
A EFFECTIVE STRESS	UGL	184.8 to 181.5	20.38	0	20
		181.5 to 180.1	19.05	0	20
		180.1 to 177.9	20.34	0	20
	HALTON	177.9 to 173.3	21.22	10	25
		173.3 to 163.0	20.60	10	25
	LGL	163.0 to 156.0	19.26	5	23
		156.0 to 152.1	18.62	0	17
	LOWER TILL	152.1 TO 149.3	BASE OF ANALYSIS		
B EFFECTIVE STRESS	UGL	184.8 to 181.5	20.38	20	30
		181.5 to 180.1	19.05	0	19
		180.1 to 177.9	20.34	8	20
	HALTON	177.9 to 173.3	21.22	20	25
		173.3 to 165.0	20.60	20	20
		165.0 to 163.0	20.60	7	29
	LGL	163.0 to 156.0	19.26	5	23
		156.0 to 152.1	18.62	0	17
	LOWER TILL	152.1 TO 149.3	BASE OF ANALYSIS		
C EFFECTIVE STRESS	UGL	184.8 to 181.5	20.38	5	20
		181.5 to 180.1	19.05	0	20
		180.1 to 177.9	20.34	5	20
	HALTON	177.9 to 173.3	21.22	10	25
		173.3 to 163.0	20.60	10	25
	LGL	163.0 to 156.0	19.26	5	23
		156.0 to 152.1	18.62	0	17
	LOWER TILL	152.1 TO 149.3	BASE OF ANALYSIS		
D EFFECTIVE STRESS	UGL	184.8 to 181.5	20.38	10	20
		181.5 to 180.1	19.05	0	20
		180.1 to 177.9	20.34	10	20
	HALTON	177.9 to 173.3	21.22	10	25
		173.3 to 163.0	20.60	10	25
	LGL	163.0 to 156.0	19.26	5	23
		156.0 to 152.1	18.62	0	17
	LOWER TILL	152.1 TO 149.3	BASE OF ANALYSIS		

* UGL - UPPER GLACIOLACUSTRINE UNIT
HALTON - HALTON UNIT
LGL - LOWER GLACIOLACUSTRINE UNIT

For this area of Canada a 0.05 seismic coefficient is suggested for design (CFEM, 1985). The probability of exceedance of this value is 10% over a 50 year time frame.

d) the presence of drains along the crest of the slope,

A drain of 5 m depth and situated parallel to the crest of the slope was incorporated into the model of the slope. This was simulated by changing the location of the phreatic surface in this area in an attempt to observe what benefits a drain might have on the stability of the upper slope.

e) the effect of tension cracks through subunit 1A.

In three of the analyses for Case B, tension cracks were assumed to extend from surface to the base of subunit 1A. The objective of these analyses was to see what influence a tension crack might have if it was to fill with water resulting in a combination in zero soil strength and an additional water pressure wedge within subunit 1A.

A summary of the strength cases is presented in Table 5.

It should be stressed that the geotechnical parameters used in the stability analyses are based on a limited number of laboratory tests on samples from a limited number of boreholes. Never-the-less, the scope of the testing is considered to be sufficient for confirming the feasibility of the conceptual design of the excavation slopes. Increasing the number and distribution of boreholes and the scope of the laboratory testing for the final design stage, may result in some modification to and refinement of the geotechnical parameters that have been adopted for the analyses presented here.

3.2.2 Basal Stability Against Upward Seepage

Basal instability of an excavation may occur if a clay deposit is underlain by a pervious stratum which is under sufficient artesian pressure to lift or heave the base of an excavation. These types of failures may be sudden if a blow-out occurs. In the case of the Preferred Site, where a wide cell base or floor will be present, the result of uplift

TABLE 5: SUMMARY OF VARIABLES IN THE SLOPE STABILITY ANALYSES

CASE	SLOPE PROFILE*			ADDITIONAL LOADING		TENSION CRACK	SURFACE DRAIN
	BENCHED & WIDTH	UPPER SLOPE	LOWER SLOPE	SEISMIC	LINE		
A-1	NO	3:1	3:1	NO	NO	NO	NO
A-2	YES 10m	3:1	3:1	NO	NO	NO	NO
A-3	YES 10m	3:1	3:1	YES	NO	NO	NO
A-4	YES 5m	3:1	3:1	NO	NO	NO	NO
A-5	YES 5m	3:1	3:1	YES	NO	NO	NO
B-1	YES 10m	3:1	3:1	YES	YES	YES	YES
A-6	YES 10m	4:1	3:1	NO	NO	NO	NO
A-7	YES 10m	4:1	3:1	NO	YES	NO	NO
A-8	YES 10m	4:1	3:1	YES	YES	NO	NO
A-9	YES 10m	4:1	3:1	YES	YES	NO	NO
A-10	YES 5m	4:1	3:1	NO	NO	NO	NO
A-11	YES 5m	4:1	3:1	YES	NO	NO	NO
B-2	YES 10m	4:1	3:1	NO	YES	NO	NO
B-3	YES 10m	4:1	3:1	NO	YES	NO	YES
B-4	YES 10m	4:1	3:1	YES	YES	YES	YES
B-5	YES 10m	4:1	3:1	NO	YES	YES	YES
B-6	YES 10m	4:1	3:1	YES	YES	NO	NO
A-12	YES 10m	5:1	2.5:1	NO	NO	NO	NO
A-13	YES 10m	5:1	2.5:1	YES	NO	NO	NO
A-14	YES 10m	5:1	3:1	NO	NO	NO	NO
A-15	YES 10m	5:1	3:1	YES	YES	NO	NO
C-1	YES 10m	5:1	3:1	YES	YES	NO	NO
D-1	YES 10m	5:1	3:1	YES	YES	NO	NO

* SLOPE PROFILE NOTED AS HORIZONTAL TO VERTICAL

pressures would not be severe and most likely would result only in some localized heaving of the base of the excavation.

The Canadian Foundation Engineering Manual, (CFEM, 1985) points out that in no event should the pore pressure at the base of the clay exceed 70% of the total stress at this point. This calculation does not take into account the shear strength of the soil and thus is considered conservative yet a good guide for design. Based on the weight of these clays and on the water pressures as determined by the finite element modelling, the proposed 15 m deep excavation is considered acceptable. However, an increase in the pore water pressure within the lower granular till would reduce the Factor of Safety associated with basal stability. Provision should be made, therefore, to monitor these pressures especially at times when the excavation reaches its full depth of 15 m.

3.2.3 Slope Stability Analysis

The Factors of Safety for the various scenarios are presented in Table 6. The modelling has considered circular arc failures of the slopes for various geometry, conditions and parameters. This has produced the related Factors of Safety for each scenario as well as the worst case or "critical failure circle". Figures showing the slope profile, minimum Factor of Safety and associated critical failure circle for each of the analyses are presented in Appendix A. A legend for the symbols used in these figures is provided at the beginning of the appendix. Table 6 should be used in conjunction with the figures in Appendix A since there is a considerable variation in the depths associated with the deep seated failure circles.

A discussion of the factors that may affect stability calculations is presented below.

a) Slope Profile Variations

There were essentially two basic slope configurations studied, an excavation without a bench and an excavation with a bench. The various benched slope scenarios were simply an attempt to optimize the configuration of the slope for the anticipated loadings and available soil strengths. The major findings of the calculations are as follows:

TABLE 6: SUMMARY OF SLOPE STABILITY ANALYSES

CASE	SLOPE PROFILE*			ADDITIONAL LOADING		TENSION CRACK	SURFACE DRAIN	FACTOR OF SAFETY				RUN ***
	BENCHED & WIDTH	UPPER SLOPE	LOWER SLOPE	SEISMIC	LINE			SHALLOW FAILURE SHORT TERM**	SHALLOW FAILURE LONG TERM	DEEP FAILURE SHORT TERM	DEEP FAILURE LONG TERM	
A-1	NO	3:1	3:1	NO	NO	NO	NO	14.5	0.8	1.4	1.2	TS1; ES1
A-2	YES 10m	3:1	3:1	NO	NO	NO	NO	16.9	1.0	1.5	1.4	TS24; ES22
A-3	YES 10m	3:1	3:1	YES	NO	NO	NO	N/A	0.8	N/A	1.1	ES23
A-4	YES 5m	3:1	3:1	NO	NO	NO	NO	17.3	1.0	1.5	1.3	TS27; ES25
A-5	YES 5m	3:1	3:1	YES	NO	NO	NO	N/A	0.8	N/A	1.1	ES26
B-1	YES 10m	3:1	3:1	YES	YES	YES	YES	N/A	1.7	N/A	1.1	ES20
A-6	YES 10m	4:1	3:1	NO	NO	NO	NO	24.0	1.2	1.5	1.3	TS2; ES2
A-7	YES 10m	4:1	3:1	NO	YES	NO	NO	N/A	1.2	N/A	1.3	ES6
A-8	YES 10m	4:1	3:1	YES	YES	NO	NO	24.0	0.9	1.2	1.1	TS6; ES16
A-9	YES 10m	4:1	3:1	YES	YES	NO	NO	N/A	N/A	N/A	1.6	ES17 ****
A-10	YES 5m	4:1	3:1	NO	NO	NO	NO	18.1	1.2	1.5	1.3	TS21; ES18
A-11	YES 5m	4:1	3:1	YES	NO	NO	NO	N/A	0.9	N/A	1.1	ES19
B-2	YES 10m	4:1	3:1	NO	YES	NO	NO	N/A	2.4	N/A	1.4	ES11
B-3	YES 10m	4:1	3:1	NO	YES	NO	YES	N/A	2.8	N/A	1.4	ES13
B-4	YES 10m	4:1	3:1	YES	YES	YES	YES	N/A	1.1	N/A	1.1	ES14
B-5	YES 10m	4:1	3:1	NO	YES	YES	YES	N/A	1.3	N/A	1.4	ES15
B-6	YES 10m	4:1	3:1	YES	YES	NO	NO	N/A	1.8	N/A	1.1	ES12
A-12	YES 10m	5:1	2.5:1	NO	NO	NO	NO	24.1	1.2	1.5	1.3	TS4; ES4
A-13	YES 10m	5:1	2.5:1	YES	NO	NO	NO	N/A	0.9	N/A	1.1	ES5
A-14	YES 10m	5:1	3:1	NO	NO	NO	NO	23.4	1.1	1.6	1.3	TS3; ES3
A-15	YES 10m	5:1	3:1	YES	YES	NO	NO	18.7	0.9	1.2	1.1	TS5; ES7
C-1	YES 10m	5:1	3:1	YES	YES	NO	NO	N/A	1.1	N/A	1.1	ES8
D-1	YES 10m	5:1	3:1	YES	YES	NO	NO	N/A	1.4	N/A	1.1	ES9

* SLOPE PROFILE NOTED AS HORIZONTAL TO VERTICAL

** FACTOR OF SAFETY FOR APPROXIMATE CORRESPONDING LONG TERM CIRCLE, NOT SHORT TERM CRITICAL CIRCLE

*** SEE APPENDIX B5 FOR GRAPHICAL RESULTS OF THE VARIOUS STABILITY ANALYSES

**** ANALYSES FOR A NON-CIRCULAR FAILURE ALONG TOP OF LOWER GLACIOLACUSTRINE UNIT

N/A NOT ANALYSED

i) Unbenched, 3:1 slopes, Case A

A 15 m deep excavation with a side slope of 3:1 (H:V) analyzed for the Case A scenario yielded long term Factors of Safety of only 0.8 and 1.2 for shallow and deep failure circles, respectively. Based on these results, if slopes were excavated at 3:1 and remained open for some time, it is probable that creep would take place and that both local shallow and deep-seated failures would develop.

The short term Factor of Safety is about 1.4 for the overall 3:1 slope suggesting a 3:1 slope configuration may be acceptable for the proposed landfill over the short term. However, these analyses neglect problems of strain softening and strength anisotropy of the deep clays and a Factor of Safety of only 1.4 may not be adequate when considering these conditions. Furthermore, the size, and timing of failures are very difficult to predict. Therefore, detailed monitoring of slopes should be of prime consideration for correlation with the excavation process to optimize the slope geometry.

ii) Benched Slopes

The need for increasing the stability of the proposed slopes was recognized and a bench was located on the slope at a depth of 5 m below existing ground surface. By benching the slope it is possible to; a) increase the stability of the slope by providing a shallower slope angle, b) provide maintenance access; and c) allow for some soil movement in the upper 5 m slope that would not interfere with the excavation and filling operations.

There is an increase in the overall stability of the slope by the provision of a bench at 5 m as shown by comparisons of the Factors of Safety between Case A-1 and Cases A-2, A-6 and A-12. However, comparing the Factors of Safety between a 5 m and 10 m wide bench under the same loading and soil strength conditions (Case A-4 vs. A-2 and Case A-10 vs. A-6) it appears that the stability

of the slope below the bench increases slightly when the width of the bench is increased.

Variation of slope angles for both the upper and lower slopes (i.e., above and below the bench) was also assessed.

The following configurations were analyzed;

3:1 slope (lower 10 m), 3:1 slope (upper 5 m) A-2
3:1 slope (lower 10 m), 4:1 slope (upper 5 m) A-6
2.5:1 slope (lower 10 m), 5:1 slope (upper 5 m) A-12
3:1 slope (lower 10 m), 5:1 slope (upper 5 m) A-14

Comparing the various configurations with one another (Table 6) it appears as if the inclination of the upper slope does not have much of an influence on the stability of either of the overall slopes or the upper slopes. The presence of water filled fissures (discussed later) does reduce the long term stability of the upper slopes by a significant extent.

b) Soil Properties (Cases A-D)

As previously discussed, four sets of soil properties ("Cases") were analyzed to establish a lower bound or lower limit for a conservative slope configuration. This was also used to determine what influence increasing or decreasing the values of these parameters would have on the calculated Factors of Safety.

For Case A, no effective cohesive intercept was assumed in Unit 1. Thus, calculated Factors of Safety less than unity were typical in the upper slope. It is apparent comparing Case A-15 with C-1 and D-1 that with any effective cohesion in this material, the long term stability of the upper slopes increases substantially.

Factors of Safety for Cases A and B were compared to observe the effects of lower limit strengths versus average soil strengths. Comparing Cases A-7 with B-2, it is apparent

that the stability of the upper slope is increased substantially in Case B, using c'=20 kPa rather than 0 kPa for the Upper Glaciolacustrine Units. Comprehensive testing to establish the c' values and field monitoring of the upper slopes to establish the time to shallow failure may be of some economic value. If any cohesion is shown to exist the upper units the upper slopes could be reassessed with the results that they possibly could be excavated at a steeper angle. The stability of the overall slope also could be modestly increased primarily due to the extra cohesion in the near surface soils.

c) Additional Loadings

Additional loadings were applied to the slope to simulate both a line of trucks on the bench and a local earthquake. Based on the calculations (A-6 vs. A-7), it appears the presence of a line of trucks on the bench could decrease the overall stability of the lower slope by approximately 6 percent. This calculation may not be relevant if trucks are finally not chosen to be used on the benches.

The occurrence of an earthquake, (A-4 vs. A-5, A-10 vs. A-11 and A-12 vs. A-13) could decrease the calculated long term stability of both the upper and lower slopes by about 25 percent. If the soil displacements (strain deformations) are large enough to cause strain softening in the sensitive lower soils, short term earthquake induced localized failures might occur (A-14 vs. A-15).

When the further testing for design has been completed, this data base should be used to evaluate the seismic aspects of the final slope designs for various periods and operational modes of the landfill lifespan. Such work is beyond the scope of the present study.

d) Surface drain

An interceptor drain constructed along the crest of the slope was analyzed to determine its effect on the stability of the upper slope. Comparing Cases B-2 and B-3 it appears as if a drain could have some beneficial long term stabilizing effects.

e) Tension Cracks

The presence of tension cracks and water pressure wedges through subunit 1A results in substantial decreases in the long term stability of the upper slopes (compare B-3 with B-5, Table 6). As indicated in Table 6, a surface drain was included in the analyses for both cases. At first thought, the presence of a drain would counteract the effects of tension cracks. However, as shown on the cross-section for Case B-5, the failure circle intersects the ground surface behind the surface drain which in turn would negate its positive effects.

3.2.4 Discussion

Based on the analyses performed to date, the following suggestions are provided with respect to the stability of the proposed 15 m deep landfill excavation.

1) The excavation slopes will have low Factors of Safety especially with respect to overall deep seated stability. Therefore, localized yielding of the slopes as a result of overstressing at and below the toe should be anticipated. The extent of yielding and the significance of strain-softening must be addressed during detailed design.

2) While the upper slopes are expected to be stable over the short term because of the preconsolidated condition of the clays, softening will occur and maintenance will be required. Surface drains properly located behind the crest to remove water from fissures and tension cracks would be of benefit in slowing the rate of softening. However, even with surface drains some water will still seep out to the face of the slope together with rain water resulting in wetting of the soil.

3) Additional loading due to stockpiles, buildings and/or berms should be situated well back from the crest of the slopes. Field loading tests would be useful to see what stockpile height is allowable so as not to damage the structure of the soils at depth which are only lightly over consolidated. The problem of pore pressures spreading laterally from stockpile areas toward the slopes should also be addressed. At the present time based on anticipated critical slip surfaces

provided by the stability analyses, a set back in the order of 16 m is tentatively suggested.

4) The final design configuration of the slopes should be determined on the basis of additional detailed geotechnical investigations carried out around the perimeter of the actual landfill excavations. Interpretation of field performance monitors, such as slope indicators, piezometers, survey pins and settlement gauges during initial excavation should be expected to cause modifications to the final slope profiles, excavation/filling schedules and the locations of stockpiles, buildings and berms.

3.3 HANDLING OF EXCAVATED MATERIALS

3.3.1 Trafficability and Workability of Materials

This section comments on the trafficability or reaction of the insitu and re-compacted clay soils to vehicular and equipment use in and around the landfill. The workability of the clays is also discussed in terms of the excavated soils recompaction rehandling and use.

The soil units which would be removed during the excavation of the landfill site to about elevation 170 m a.s.l. (15 m depth) are the Upper Glaciolacustrine Unit (Unit 1) and the upper third of the Halton Unit (Unit 2). Typically the Upper Glaciolacustrine Unit extends from ground surface to an elevation of about 178 m a.s.l. and the underlying Halton Unit extends to an approximate elevation of 163 m a.s.l.

Both units can be described generally as clayey silts of low to medium plasticity, however, a layer of highly plastic glaciolacustrine clay was encountered in Unit 1 at 3-5 m depth. Due to preconsolidation, many of the soils have a moisture content near their plastic limit, at least to a depth of 5 m below existing ground surface. Below this depth the clayey silts have natural moisture contents more or less wetter than their plastic limit as shown by shaded zone on Figure 2-1.

The sensitivity values for the clayey soils (measured by laboratory vane tests) varied from 5 to 20 with one high value of about 32. The soils, therefore, are considered to be

of low to medium sensitivity (CFEM, 1985). These measured sensitivity values are considerably higher than those reported by Conlon et al, 1971 and seem to be somewhat incompatible with Liquidity Index values of 0.1 to 0.5 obtained from the soils at the Preferred Site. The reasons for this behavior should be clarified during the next phase of testing.

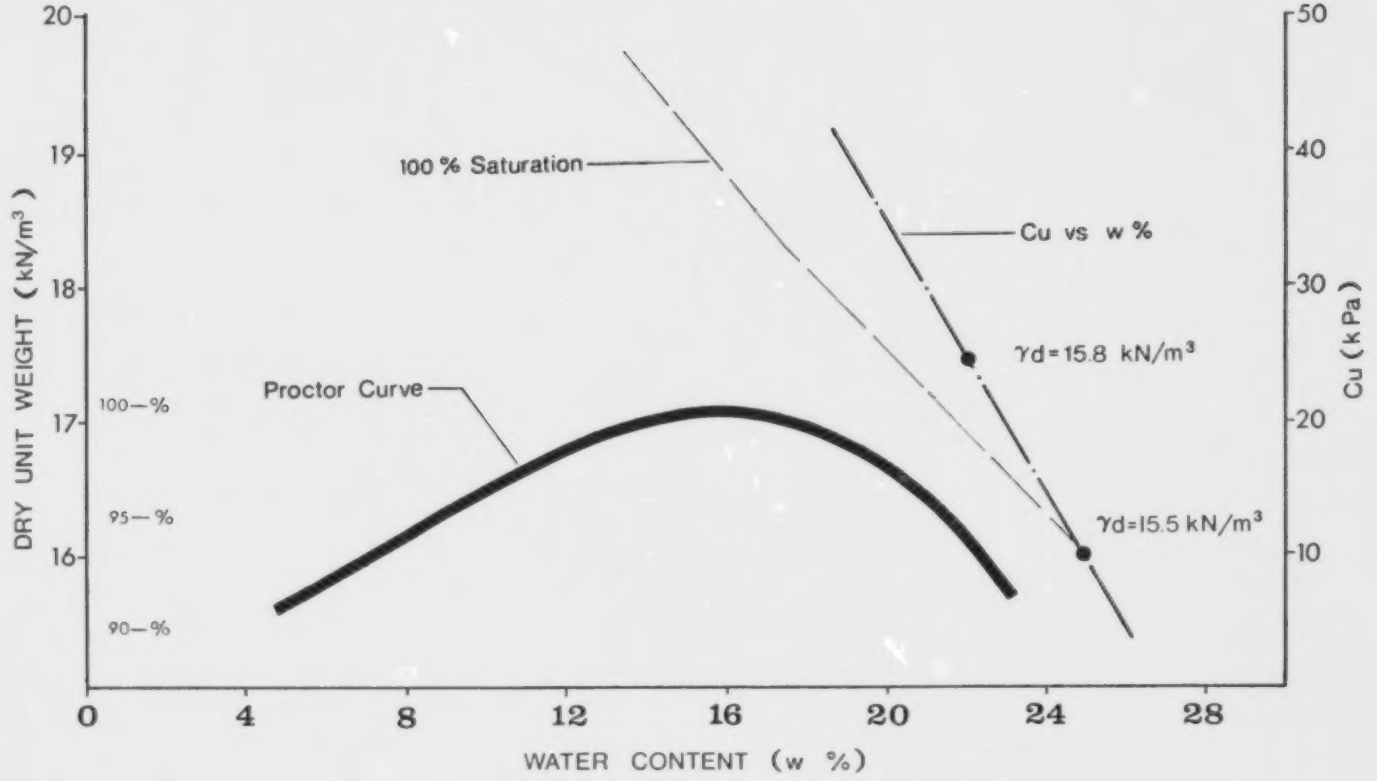
Two compaction tests were carried out on composite samples from the Upper Glaciolacustrine and Halton Units as shown on Figures 3-2 and 3-3, respectively. Two important points are illustrated on these figures, 1) the natural moisture contents may be significantly wetter than optimum and 2) the undrained shear strength of the remoulded soils at these high water contents is very low.

The combination of wetter than optimum moisture conditions and relatively high sensitivity suggests that both the Upper Glaciolacustrine and Halton Units will have fair to poor workability and trafficability characteristics. Additional compaction tests are required to define the behavior of these soils, especially since the degree of preconsolidation varies so much within the 15 m excavation zone. Should the soils prove to be less sensitive than observed in this study, they would be easier to handle.

Trafficability and workability are further influenced by seasonal weather conditions. During wet periods both the in-situ and recompacted soil will be wetter and more plastic and thus will be more difficult to work on.

During excavation of the Welland Canal By-Pass and the Townline Road rail tunnel, a number of problems arose while excavating similar materials. Owen (1969) and Christensen (1987) noted that scrapers were successfully used to remove the upper desiccated material however, as the softer material was encountered below 3 or 4 m depth, draglines were required to effectively excavate the clayey silt. These problems are inferred to have occurred at natural moisture contents about 5% above the plastic limit and possibly at sensitivities as low as 3 or 4.

FIGURE 3-2
Standard Proctor Compaction Test Results - Upper Glaciolacustrine Unit



Natural Water Content 29%

Range of Natural Water Content 17 to 33%

Optimum Water Content 16.4%

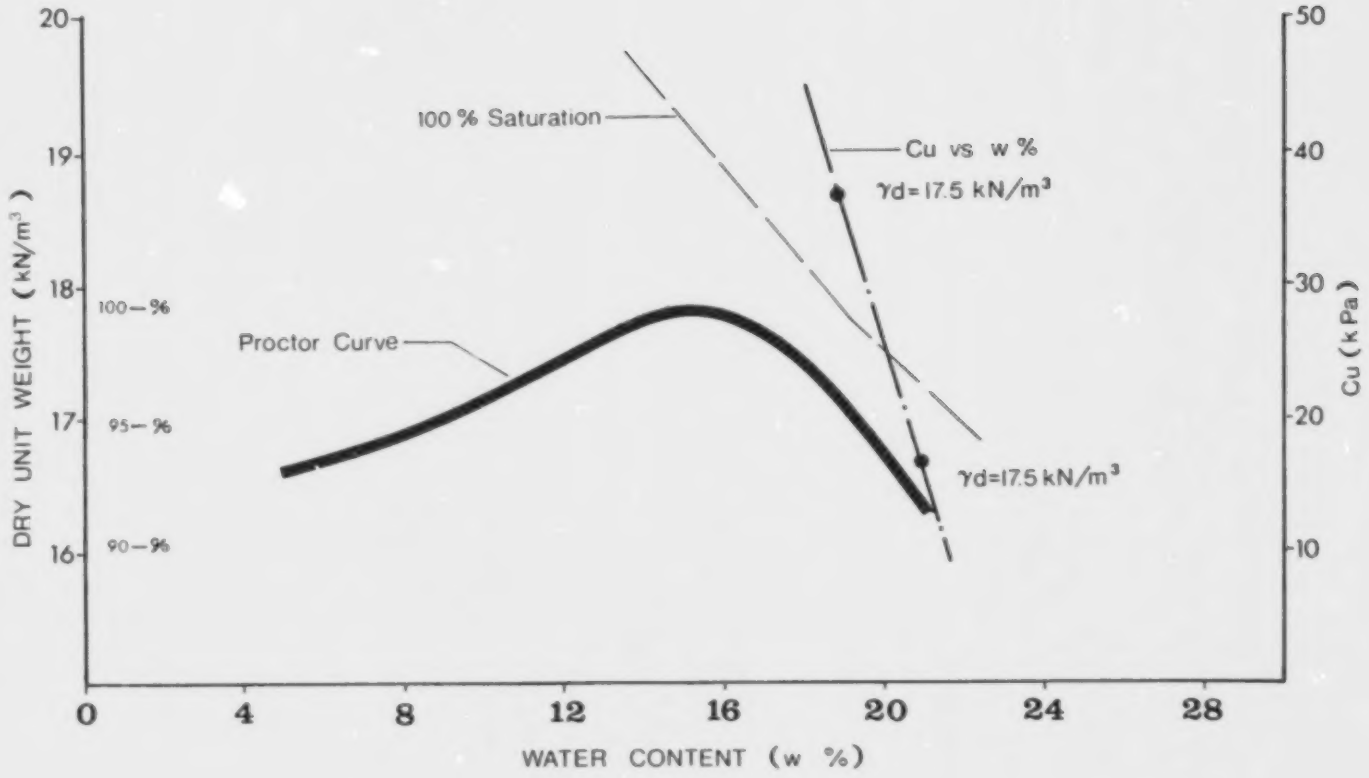
Maximum Dry Unit Weight 17.12 kN/m3

● Undrained Shear Strength of soil compacted at water content shown

γd Corresponding Dry Unit Weight

(Figure taken from Appendix E1; GLL , 1987a)

FIGURE 3-3
Standard Proctor Compaction Test Results - Halton Unit



Natural Water Content 21%

Range of Natural Water Content 17 to 27%

Optimum Water Content 15.6%

Maximum Dry Unit Weight 17.85 kN/m3

● Undrained Shear Strength of soil compacted at water content shown

γd Corresponding Dry Unit Weight

(Figure taken from Appendix E1; GLL , 1987a)

The following suggestions are made with respect to the workability and trafficability of Units 1 and 2:

- wide tracked equipment with low ground contact-pressures should have the best mobility on this type of soil as long as it remains dry and relatively undisturbed. (i.e., bulldozers, scrapers and draglines);

- handling of soil materials may have to be limited to the drier months although frozen ground excavation may be feasible depending on the excavation procedure;

- all weather roads should be available for construction traffic on site;

- minimum handling of the soils is recommended to avoid excessive strength loss due to the apparent sensitivity of the soils below 5 m depth;

- efficient routing plans patterns and circulation schedules for equipment would limit the number of construction and access roads on site and minimize soil disturbance associated with repetitive loading especially near the base of the excavation; and

- estimated expansion factors upon excavation of the silty clay material will depend on the extent of disturbance created by handling the generally sensitive soil. The more preconsolidated the soils, the greater the expansion factor, which could vary from zero for a slurry to 20 or 30% for the desiccated clay.

3.3.2 Stockpiling of Materials

The current design of the facility assumes that some of the clayey silt materials excavated from the landfill may be stockpiled on-site and used later to construct berms and/or landfill covers (Monenco, 1987). As previously noted the moisture contents and apparent sensitivity of a large portion of these soils, suggest that they will easily remold and thus will have fair to very poor handling characteristics.

Large stockpiles up to 8 m high may be located on site depending on landfill development considerations. It is further understood that these stockpiles may have life spans in the order of 20 years and may be comprised largely of uncompacted material. Preliminary calculations, based on estimated effect shear strength parameters (c'=0, phi res=21 degrees), indicate the clay stockpiles may not be stable at 3:1 (H:V) side slopes given the above scenario. Furthermore, compactive efforts aimed at increasing the stability could result in high pore pressure build-up, due to over compaction, causing short term or construction slope failures within the stockpiles. Railway and roadway overpasses constructed of excavated local clays during the Welland Canal realignment (Christensen, 1987), experienced short term (during construction) failures due to the low shear strengths of the compacted soils. The use of toe berms constructed of the drier surface soils could be considered for use around the stockpiles.

Another very important consideration is the combined effect of pore water pressure build-up combined with strain softening on the bearing capacity at depth beneath the stockpile. If the stockpiles are too close to the excavation slopes, spreading pore pressures could possibly affect the overall stability of the landfill slopes.

To improve the physical behavior of the stockpile materials, the following suggestions are provided:

- placement of the excavated soils should be during non-freezing dry weather to minimize increases in the water content of the clay;
- on site inspection by qualified geotechnical engineers during excavation and placement is recommended;
- compaction of the excavated material should be carried out using equipment which will cause minimum remoulding of the more sensitive soils yet achieve an adequate compactive effort, and
- surface erosion of the silty soils would be minimized during or as soon as practical after construction.

The stability of the excavated sideslopes in working areas of the landfill must not be affected by the presence of the clay stockpiles. Calculations for the required "set back" from the edge of the excavation should be completed during the design phase. Field monitoring of strategically placed piezometers may be required during field operations, to confirm the calculations.

Calculations to date indicate that the stresses resulting from an 8 m high, broad clay stockpile will approach the effective preconsolidation pressures of the Lower Glaciolacustrine Unit. Maximum and differential settlements caused the clay stockpiles may be influenced by the effects of over stressing of the soil fabric at depth. Construction of clay stockpiles above completed landfill areas should be kept low enough to prevent overstressing and settlement at depth which might cause differential settlements of the proposed drainage system.

Before the design and placement of the on-site stockpiles final additional detailed geotechnical studies are required.

3.4 PLACEMENT OF COVER MATERIALS

It is understood that the stockpiled clayey silt material excavated from the landfill site will be used as cover material over the solidified mass. A preliminary review of the cover materials available and the potential placement techniques has been completed and are briefly discussed below.

Due to the feasibility nature of the work, compaction testing was limited to two composite soil samples as part of the geotechnical component of the Phase 4B study. (See Section 3.3.1). Therefore, it is difficult to establish any rigid specifications for the cover materials at this time. The testing has established a basis for general comments and guidelines. In particular, the permeability of the cover materials will be a function of the soil units used, and the amount of remoulding created by subsequent placement techniques. The following general apply to the placement of engineered covers:

- The upper 5 m portion of the Upper Glaciolacustrine Unit and possibly the upper portion of the Halton Unit, both with in-situ moisture contents near their plastic limits, may be the easiest soils to compact for cover materials. However, depending on the excavation techniques employed, the selective separation during excavation of the Upper Glaciolacustrine and the Halton Unit may not be practical. Therefore, the cover materials may be comprised of a heterogeneous mixture of Units 1 and 2. Scraper excavation and stockpiling, of the top 5 m of soil might be considered if this soil is proven in later studies to be the most suitable.

- On site inspection by qualified geotechnical engineers is recommended for construction control during excavation and placement of cover materials.

- The compactive effort and placement technique will be influenced by the design permeability of the cover. Since only two compaction tests have been carried out to date, it is necessary that additional testing be done to develop specifications. An assessment of the workability of the excavated soil by field trials is also recommended before placement specifications of the cover are finalized.

- If the final lift of the cover is not placed during the construction period, the already installed material must be protected over the winter months from frost action.

- If possible, the placement of the cover should be in the non-freezing drier months to control the water content of the clay. Based on a preliminary study of environmental conditions at the selected site (Squires, 1986), it appears the operating schedule for such work will be six months in duration and will be subject to modification depending on weather conditions.

3.5 SURFACE WATER AND GROUND WATER CONTROL

An increase in the moisture content of the silty soils will impair their handling and trafficability characteristics. As a result the interception and diversion of surface water

from the excavation via a network of drainage ditches is of considerable importance during construction and operations of the landfill.

As the surface soils are quite stiff, the proposed diversion ditches can be excavated to any reasonable depth. It is understood the final depth and cross-sectional areas of the channels will be established primarily from hydrological data rather than geotechnical considerations.

The depth of ditches may be limited somewhat by the long term side slope stability since there is a highly plastic zone at an approximate elevation of 182 m a.s.l. (approximately 3 m below existing grade). Provided the ditches are no deeper than 1.5 m, it is suggested that side slopes of the ditches be maintained at 3:1 (H:V) to no steeper than 2:1 (H:V) depending on site conditions. These channels should be protected from erosion as soon as practical, by some form of lining and/or vegetation.

Ground water seepage into the excavations is expected to be minimal due to the low permeability of the clayey silt units. The dewatering capacity of the sumps should be related to anticipated rain and snowfall quantities from both normal and unusual conditions.

4.0 GENERAL FACILITY DESIGN AND CONSTRUCTION CONSIDERATIONS

4.1 INTRODUCTION

During Phase 4B a number of geotechnical issues arose related to the construction of the proposed facilities at the Preferred Site. A general discussion of these issues is presented below. Site specific geotechnical investigations will be required for final design.

4.2 HANDLING OF EXCAVATED MATERIALS

The handling characteristics of the materials have been discussed in Section 3.3 and the reader is referred to this section for details.

4.3 FOUNDATIONS

A variety of structures are planned for the proposed facility. All facilities should be designed in accordance with the National Building Code of Canada, including an allowance for earthquake loading.

The following points are presented to provide some guidance for preliminary foundation design:

- amount of overconsolidation and the shear strength of the soils both tend to decrease with depth. Design of spread and raft foundation is an alternative to prevent overstressing at depth where the soil is particularly sensitive;

- a number of shallow, poorly drained swales occur throughout the proposed Central Operating Area. Based on the information presented in Section 2.0 of the GLL, 1987a report, it appears the sediments in the swales are approximately 1.5 m thick and are very soft to firm in consistency. As a result of this variability these areas require specific investigations prior to final foundation design;

- If end bearing piles are used to support any heavy structures on-site, problems with pore pressure build up and migration should be addressed relative to slope

stability. Also downdrag that would create additional loads on the piles caused by settlement should be considered; and

- sulphate concentrations (GLL, 1987a) within the shallow overburden ground water flow system (above about 9 m depth) were found to have an excess of 5000 mg/L. Below 9 m, sulphate concentrations were in the order of 400 mg/L. Measures to control corrosion of metals and deterioration of concrete will probably be required.

4.4 DRAINAGE DITCHES

Construction considerations for drainage ditches have been discussed in Section 3.5 and the reader is referred to this section for details.

4.5 BERMS

Some clayey silt materials excavated from the landfill will be used to construct berms in and around facility.

Since the Upper Glaciolacustrine unit is heavily overconsolidated and has water contents close to the plastic limit, it is anticipated berms may be constructed at 3:1 (H:V) side slopes, depending on site conditions and materials used. Additional compaction tested and strength would be required to confirm the final design of the berms.

4.6 ROADWAYS

Some on-site roadways will be required for both construction vehicles and later by transporting trucks. The following section provides insight to the pavement soils design of Highway 20 to the north of the Preferred Site.

During the reconstruction and widening of Highway 20 from Bismark to the east, the Ministry of Transportation and Communications recommended that following pavement design; 200 mm crushed clean 19 mm stone, overlain by a minimum of 250 mm of granular "A" and 120 mm of asphalt. All required grade raises were to be constructed using an increased granular "A" thickness (Sturm, 1986). In the vicinity of

the Preferred Site the widened road beds were comprised of a 0.6 m base course of granular "A" (19 mm crushed) and approximately 120 mm of asphalt (Davis, 1986). The substantial granular "A" thickness was based on the fact that there is a lack of granular "B" and "C" in that area of the Peninsula and that the costs associated with increasing the granular "A" was less than obtaining crushed clean stone for the sub-base. The thick granular base course was also designed to traverse the deformable clayey silt soils in the area which can be locally frost susceptibile.

Prior to final design, additional on-site investigations will be required along the confirmed alignments to finalize the pavement design.

APPENDIX A

SLOPE STABILITY ANALYSIS PLOTS - SUPPORTING DATA

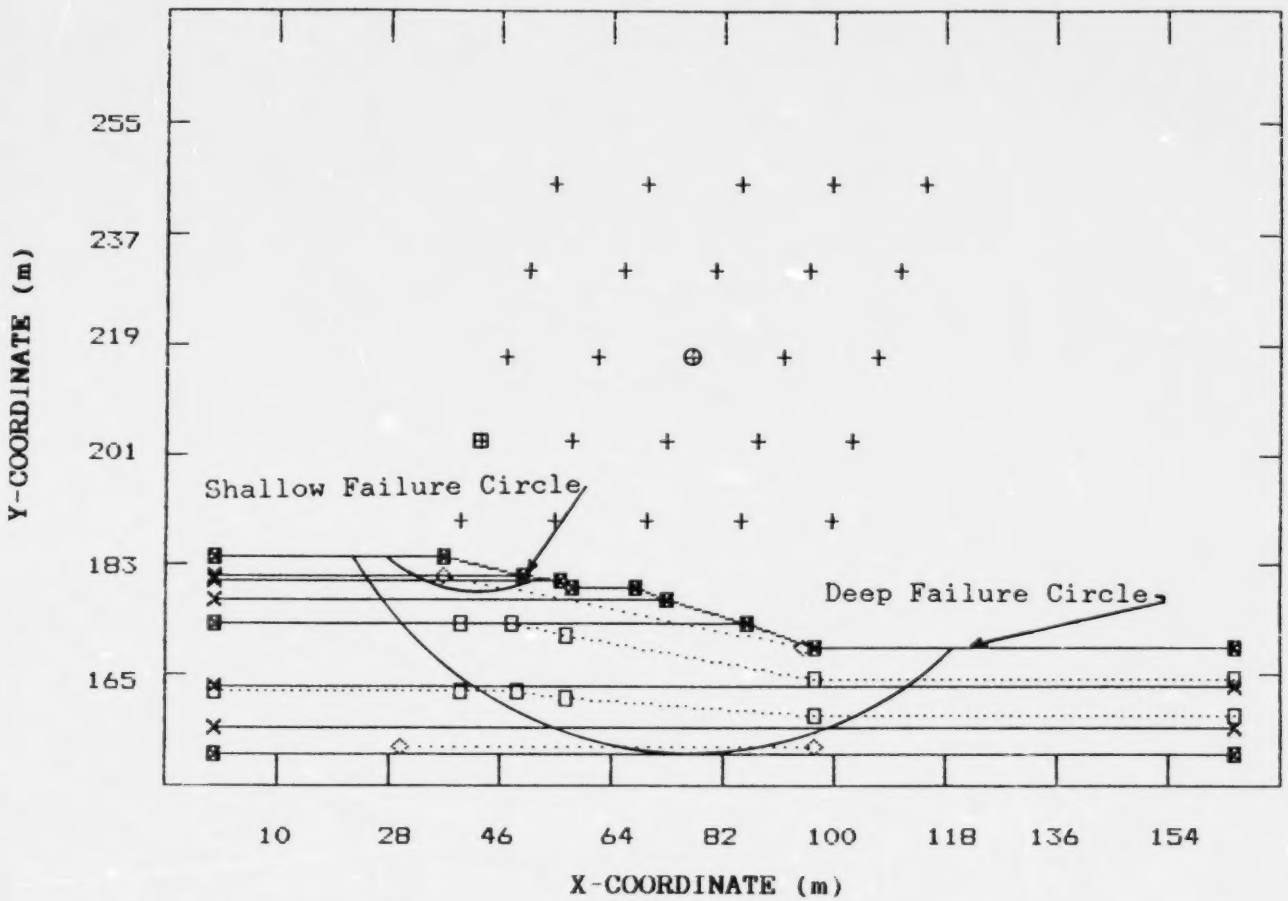
The results of the slope stability analyses are presented in the appendix. For each analysis, the values of unit weight, cohesion and internal angle of friction are summarized on the geometry plots. Also presented for each slope analyzed are the contacts between each subunit, the pore pressure contours, centre of each trial circle, and the critical failure circle.

The figure entitled Legend for Cross-Section of Geometry Plots explains how these data are presented in the subsequent figures.

Appendix A1 and A2 present the results of the Total Stress and Effective Stress analyses, respectively.

LEGEND FOR CROSS-SECTION OF GEOMETRY PLOTS

TITLE OF PROJECT
RUN NUMBER DATE
TOTAL STRESS OR EFFECTIVE STRESS ANALYSES



SOIL PROPERTIES USED FOR ANALYSES

UNIT WEIGHT (kN/m³)	COHESION (kPa)	PHI (Degrees)	PHI B (Degrees)	
20.38	0.0	20.0	0.0	SUBUNIT 1A
19.05	0.0	20.0	0.0	SUBUNIT 1B
20.34	0.0	20.0	0.0	SUBUNIT 1C
21.22	10.0	25.0	0.0	SUBUNIT 2A
20.60	10.0	25.0	0.0	SUBUNIT 2B
19.26	5.0	23.0	0.0	SUBUNIT 3A
18.62	0.0	17.0	0.0	SUBUNIT 3B
-1.00	0.0	0.0	0.0	UNIT 4

(Case Number)

\+ Centre of Trial Circles with Corresponding Factor of Safety
x—x Stratigraphic Contact
□—□ Pore-Pressure Contour
◇—◇ Tangent of Trial Circles
□ Centre of Critical Shallow Failure
○ Centre of Critical Deep Failure

APPENDIX A1

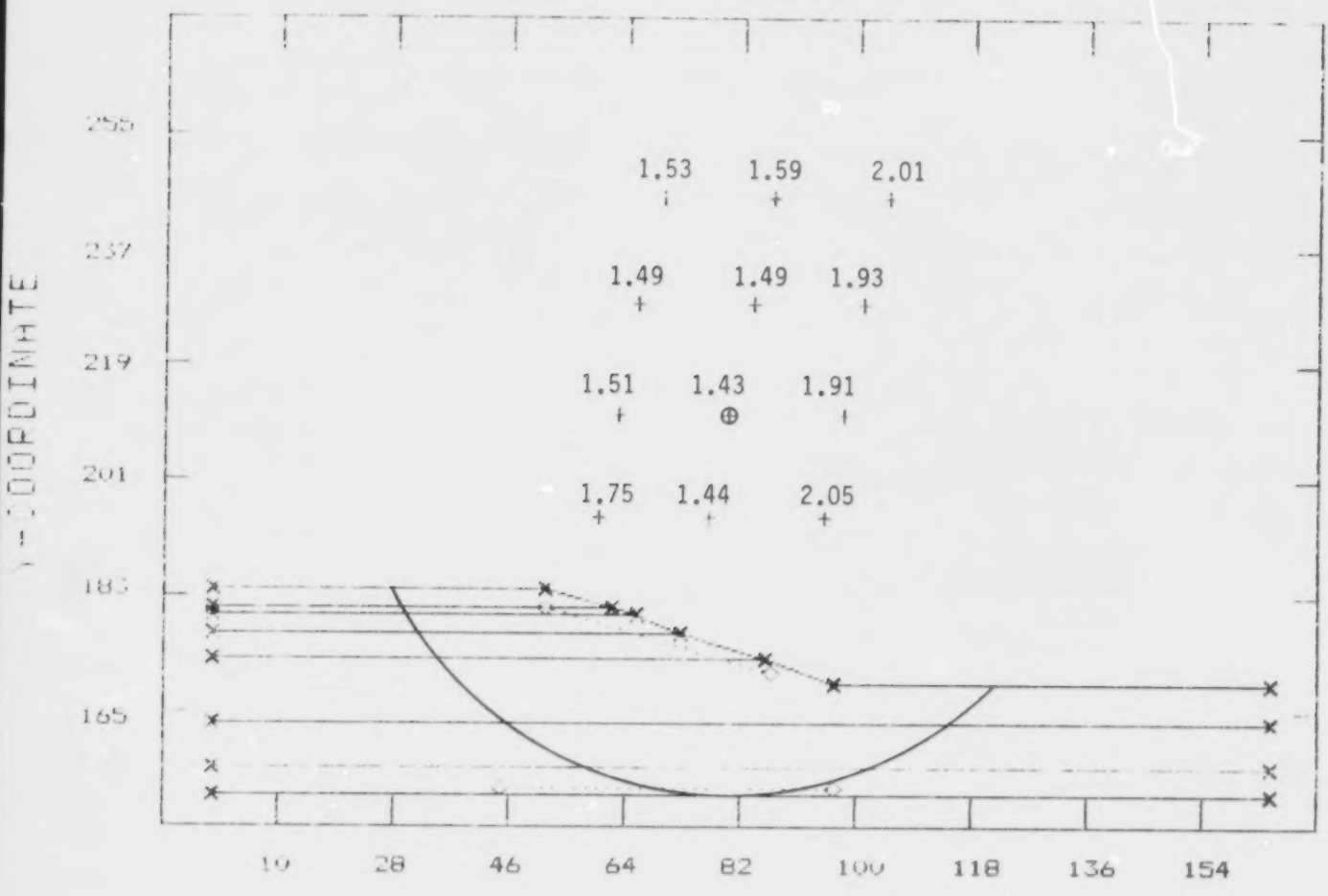
TOTAL STRESS ANALYSIS PLOTS

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B, GEOTECH 85-GT-5

1 JUNE 30, 1986

TOTAL STRESS ANALYSIS.



UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	80.0000	.0000	.0000
20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2600	61.0000	.0000	.0000
18.6200	61.0000	.0000	.0000
1.0000	.0000	.0000	.0000

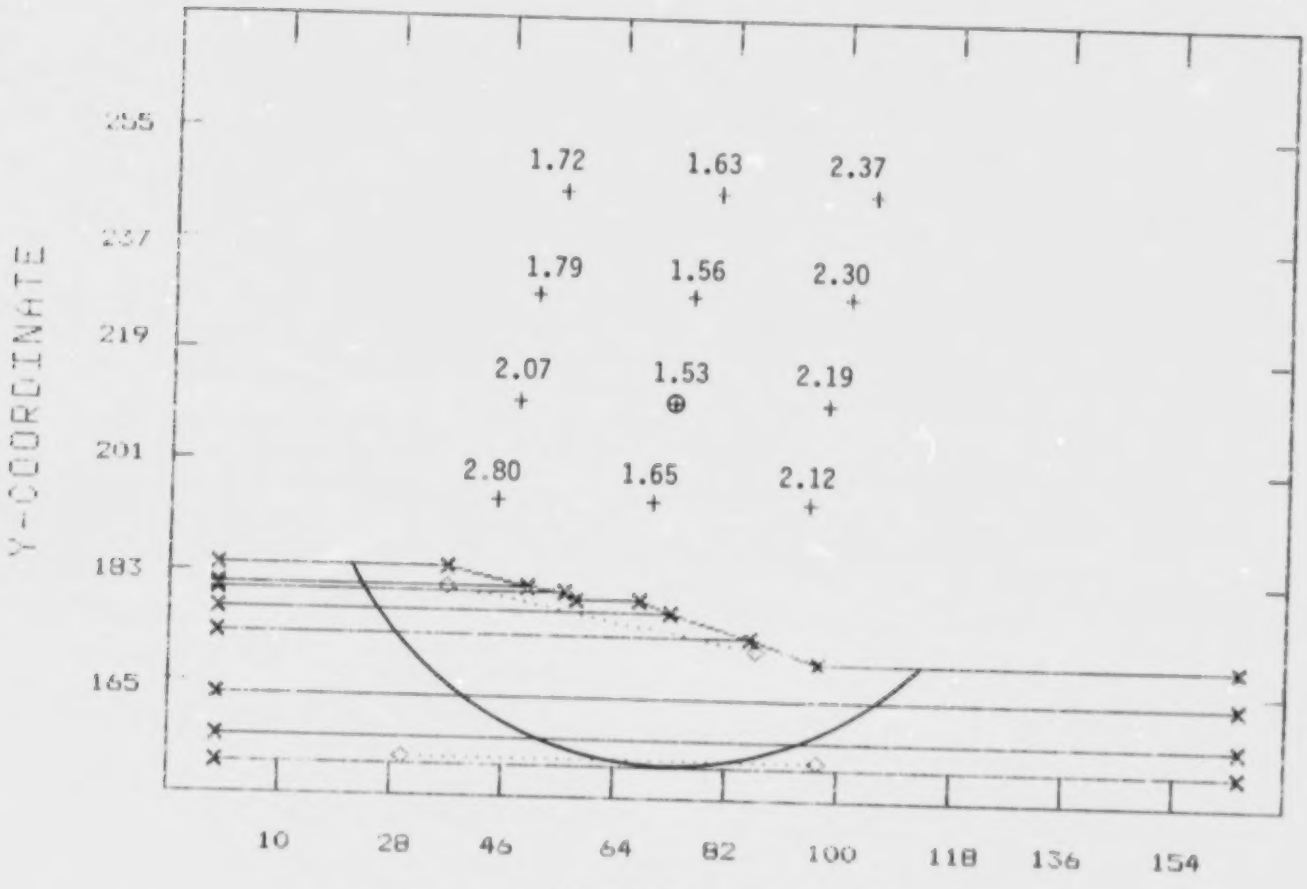
(Case A-1)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B, GEOTECH 85-GT-5

2 JULY 2, 1986

TOTAL STRESS ANALYSIS,



UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	80.0000	.0000	.0000
20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2800	81.0000	.0000	.0000
18.6200	61.0000	.0000	.0000
-1.0000	.0000	.0000	.0000

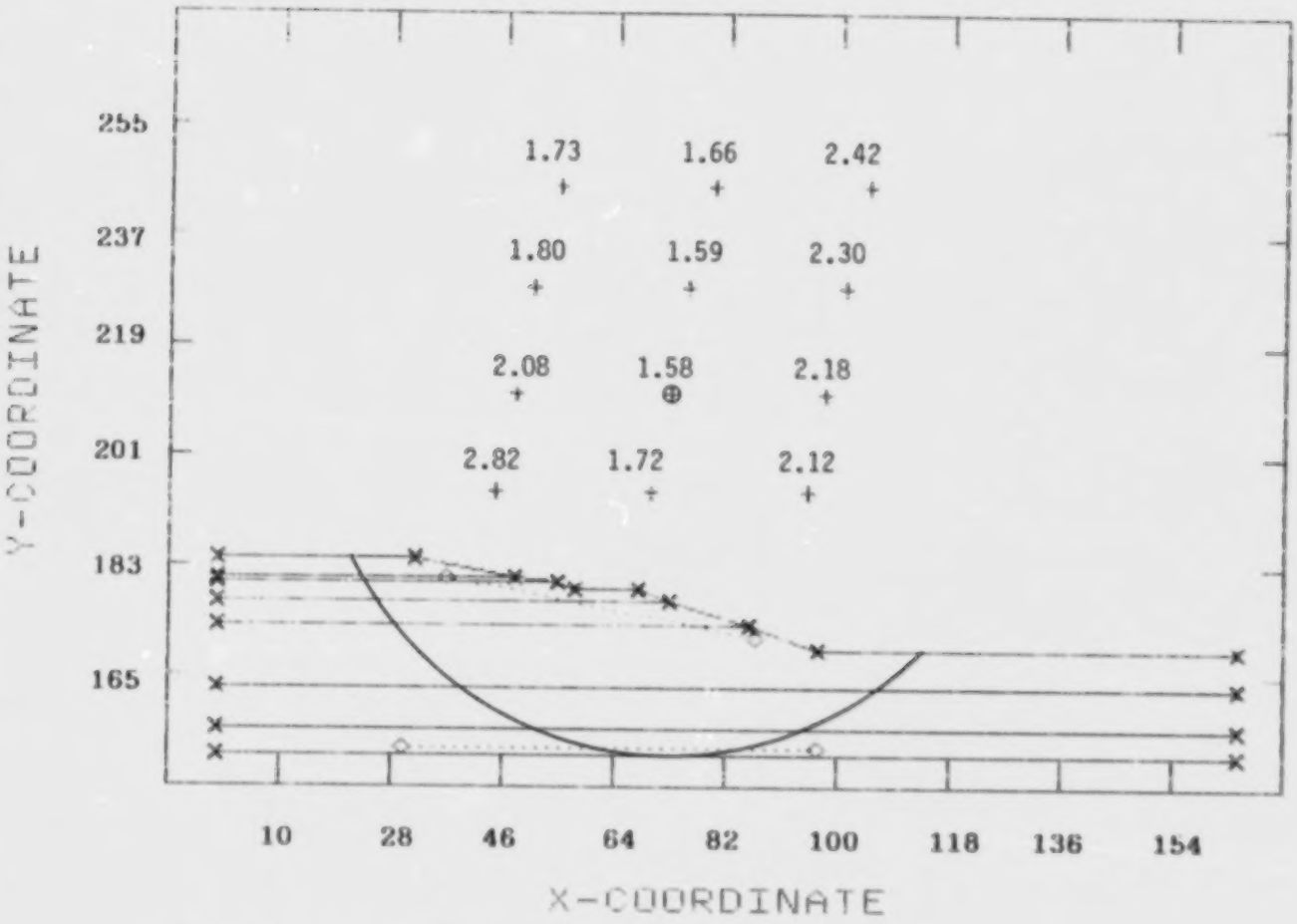
(Case A-6)

CROSS-SECTION OF GEOMETRY

OWMC PHASE, 4B GEOTECH 85-GT-5

3 JULY 2,1986

TOTAL STRESS,



UNIT WEIGHT	COHESION	PHI	PHI B
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20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2600	61.0000	.0000	.0000
18.6200	61.0000	.0000	.0000
-1.0000	.0000	.0000	.0000

(Case A-14)

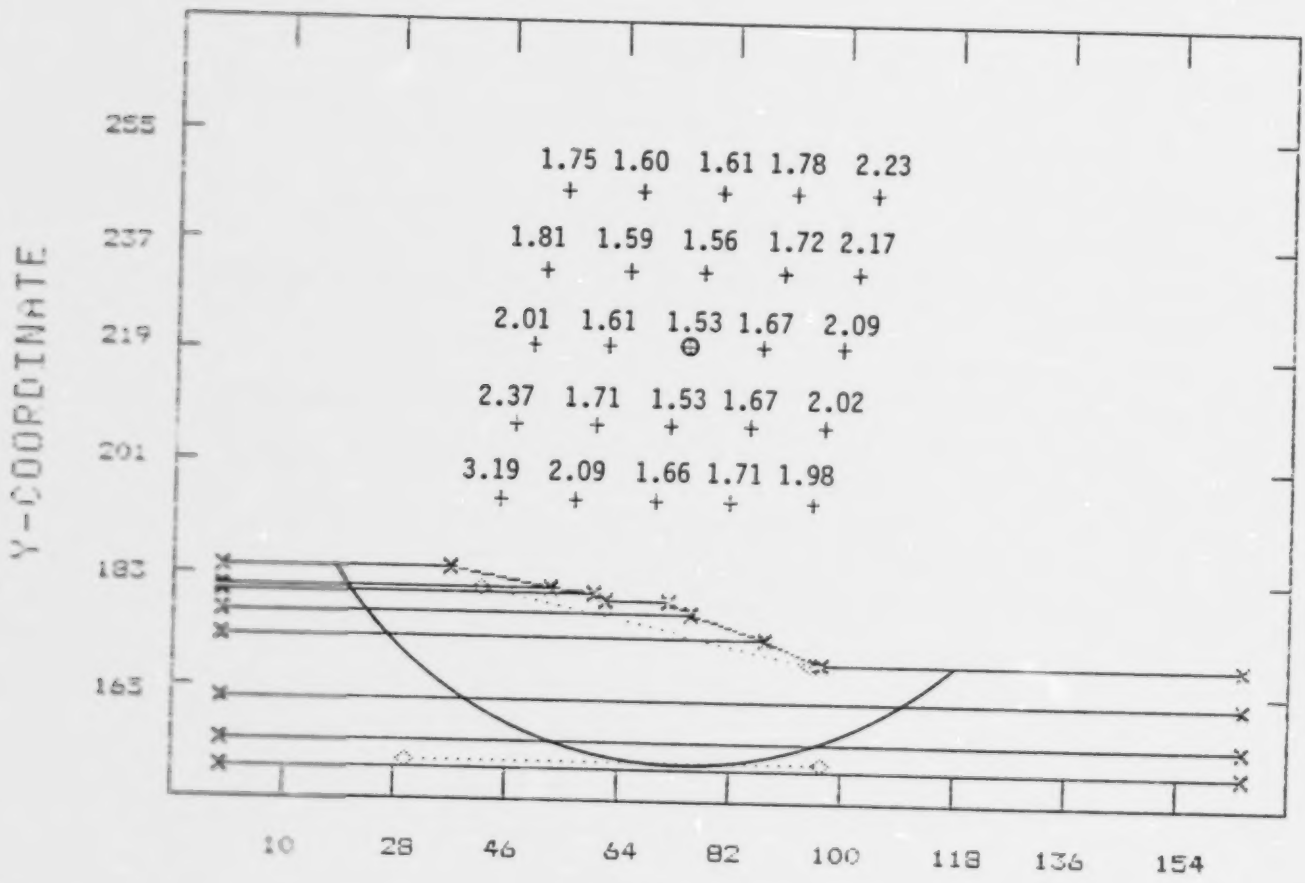
CROSS-SECTION OF GEOMETRY

OWMC, PHASE 4B, GEOTECH 85-GT-5

4

TS.

JULY 16, 1986



UNIT WEIGHT	COHESION	PHI	PHI B
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20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2600	61.0000	.0000	.0000
18.8200	61.0000	.0000	.0000
-1.0000	.0000	.0000	.0000

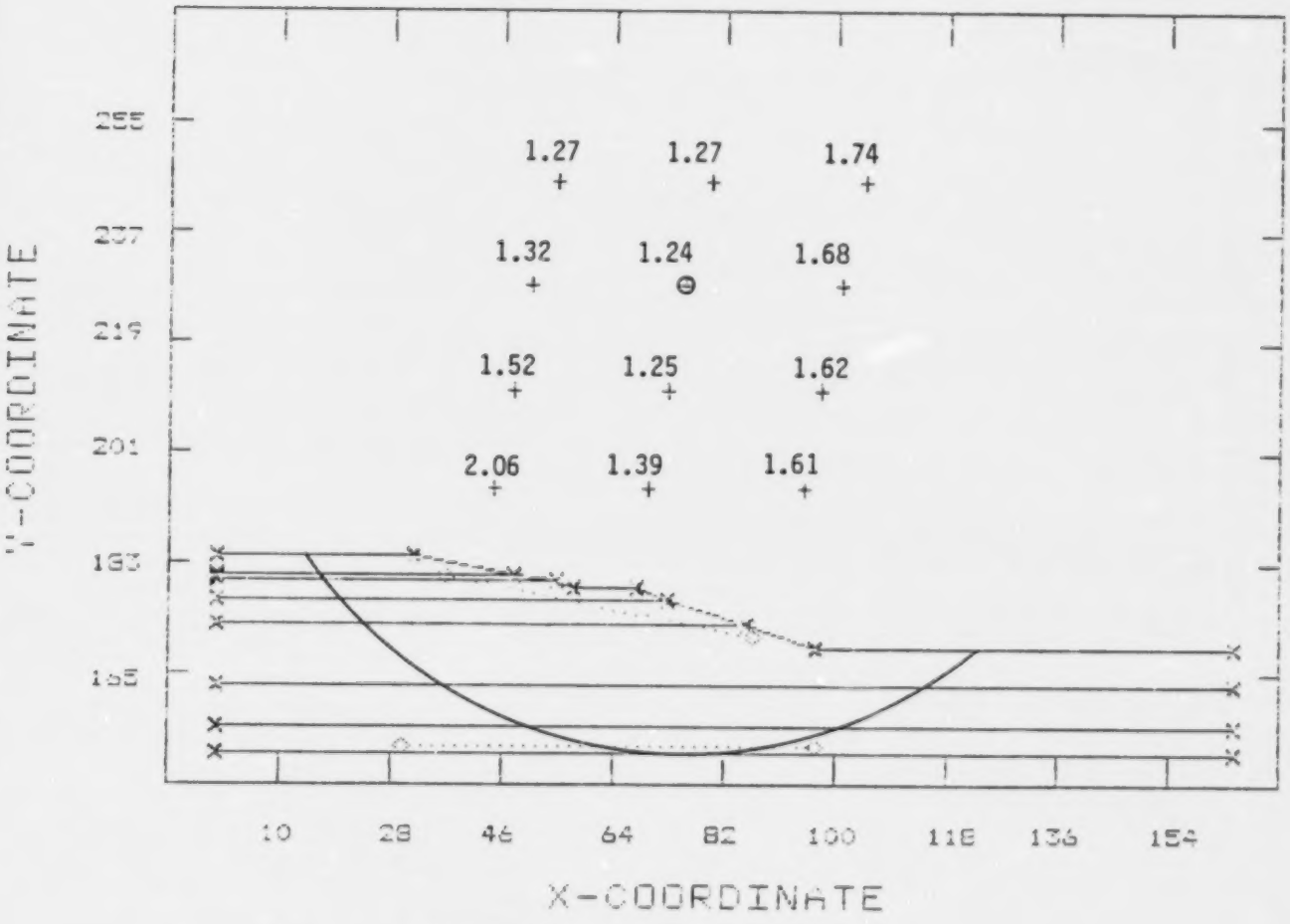
(Case A-12)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, GS-GT-S

5 JULY 17, 1986

TS,



UNIT WEIGHT	COHESION	PHI	PHI B
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20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2600	61.0000	.0000	.0000
18.6200	61.0000	.0000	.0000
-1.0000	.0000	.0000	.0000

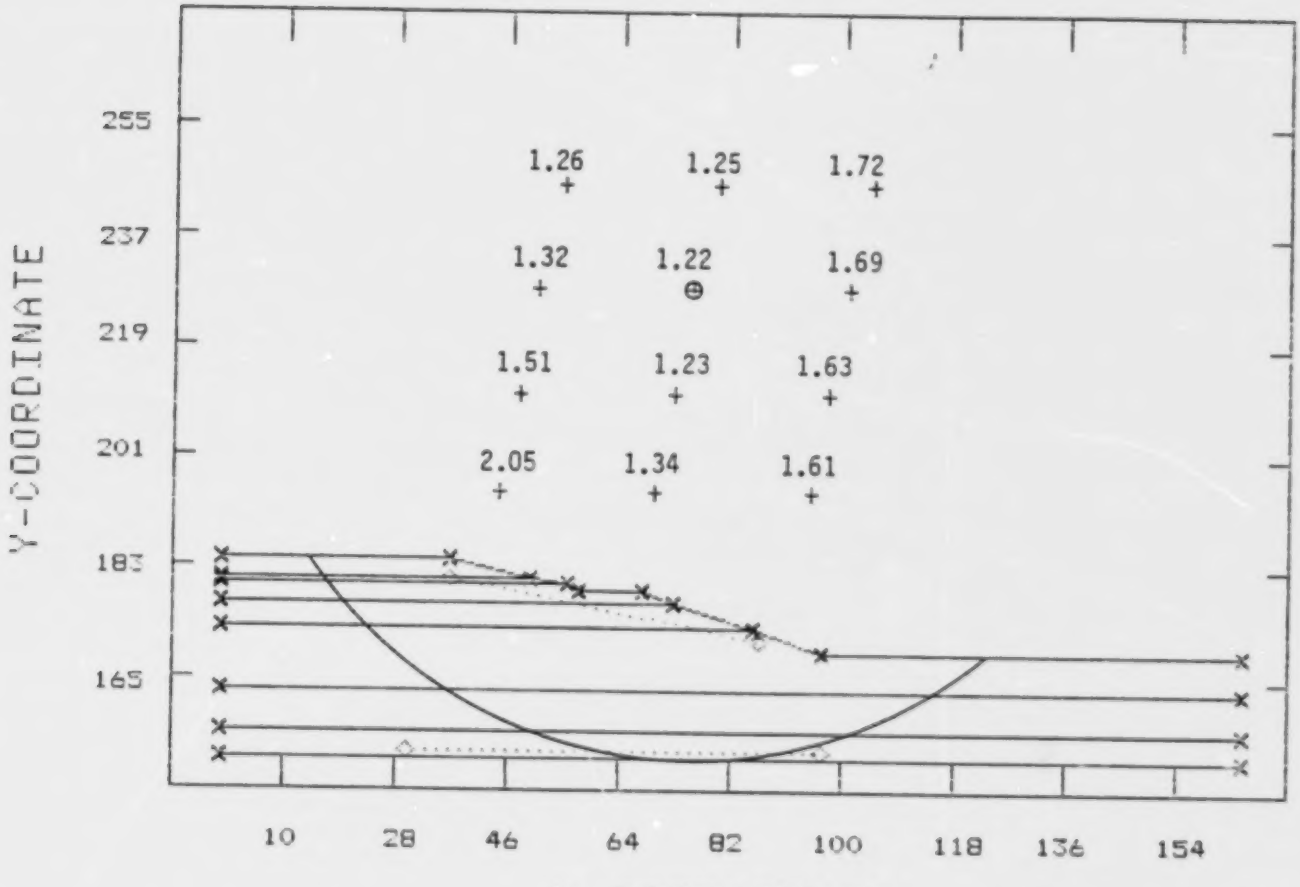
(Case A-15)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

6 AUGUST 26, 1986

TS,



UNIT WEIGHT	COHESION	PHI	PHI B
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20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2600	61.0000	.0000	.0000
18.6200	61.0000	.0000	.0000
-1.0000	.0000	.0000	.0000

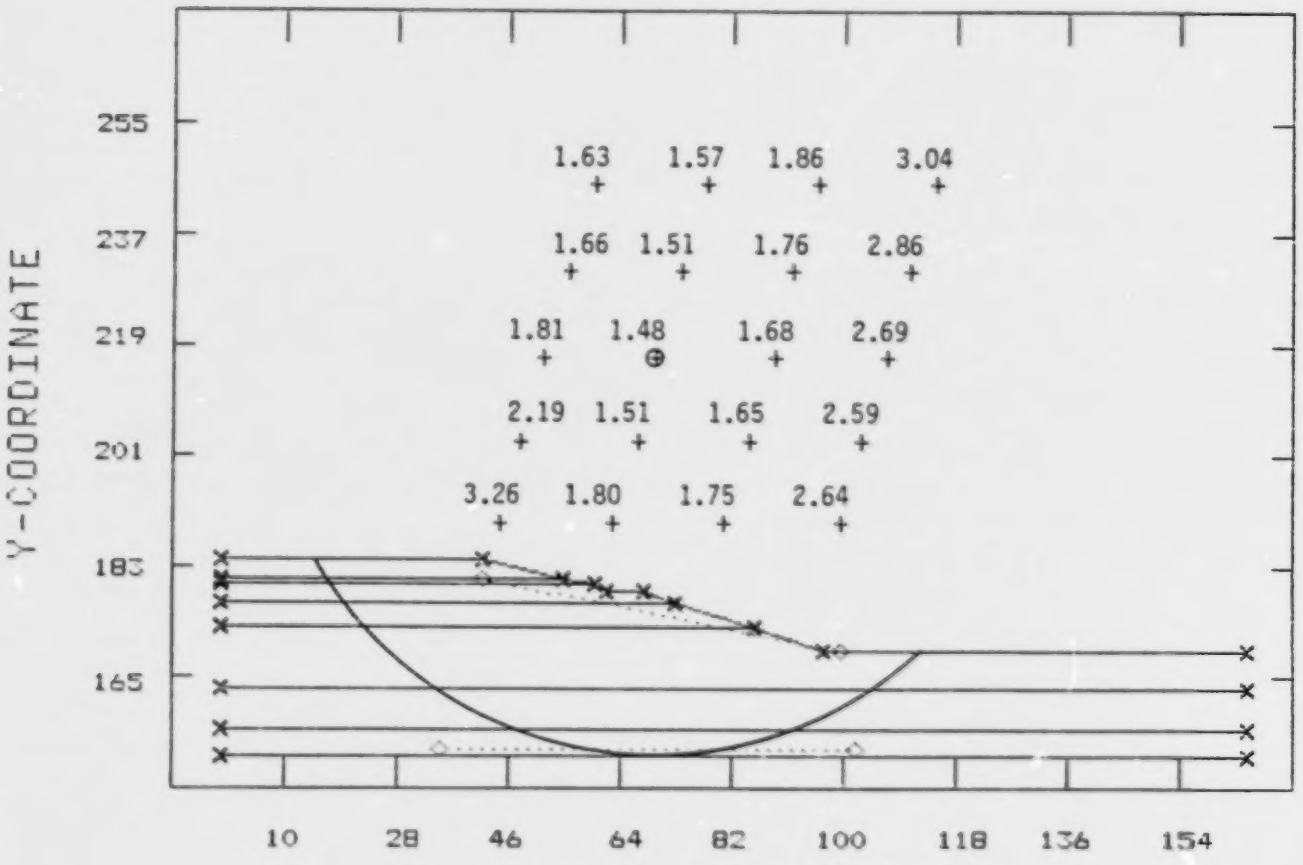
(Case A-8)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

21 OCT 27,1986

TS,



UNIT WEIGHT	COHESION	PHI	PHI B
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20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2600	61.0000	.0000	.0000
18.6200	61.0000	.0000	.0000
-1.0000	.0000	.0000	.0000

(Case A-10)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

24 OCT 27, 1986

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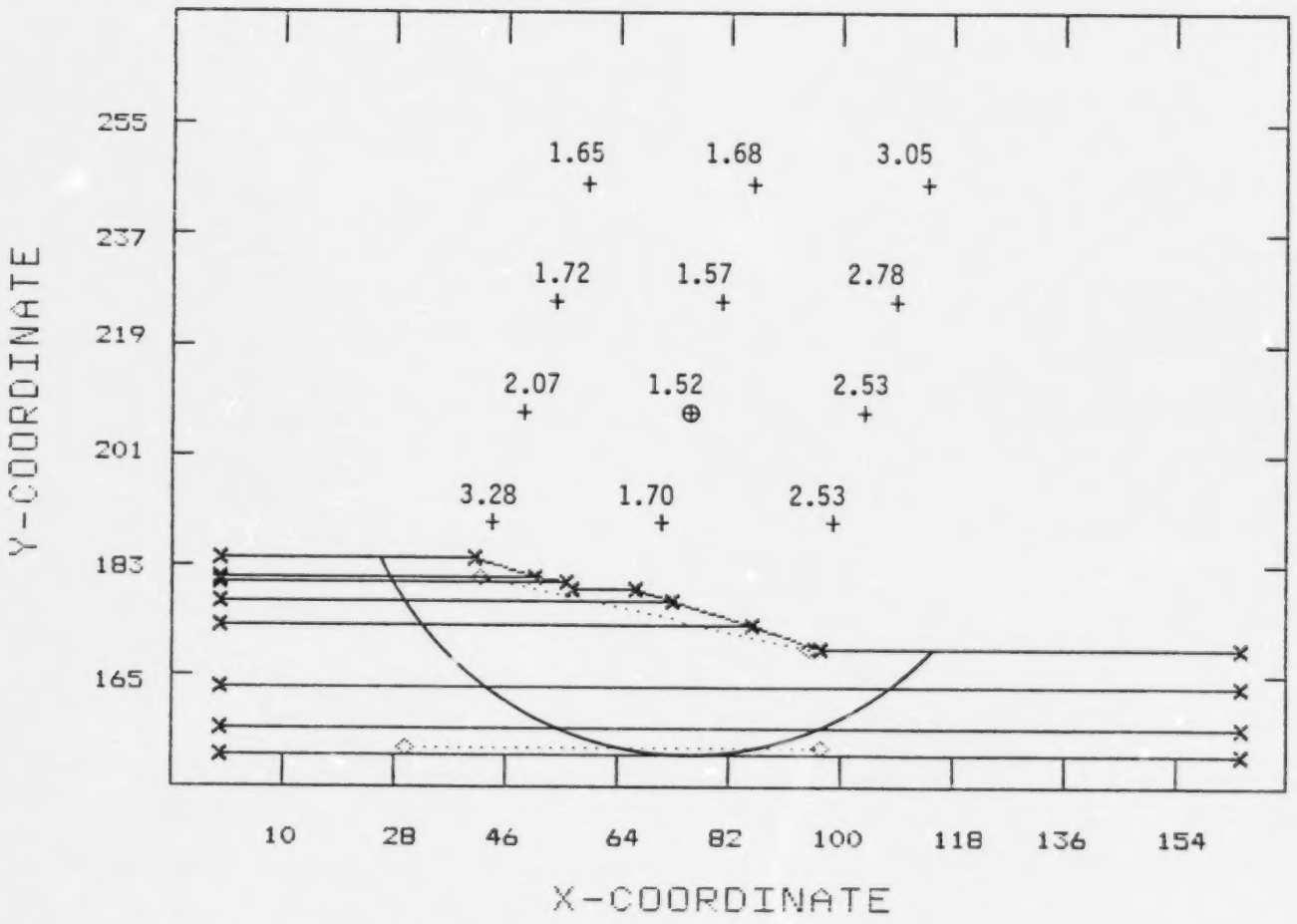

UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	80.0000	.0000	.0000
20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2600	61.0000	.0000	.0000
18.6200	61.0000	.0000	.0000
-1.0000	.0000	.0000	.0000

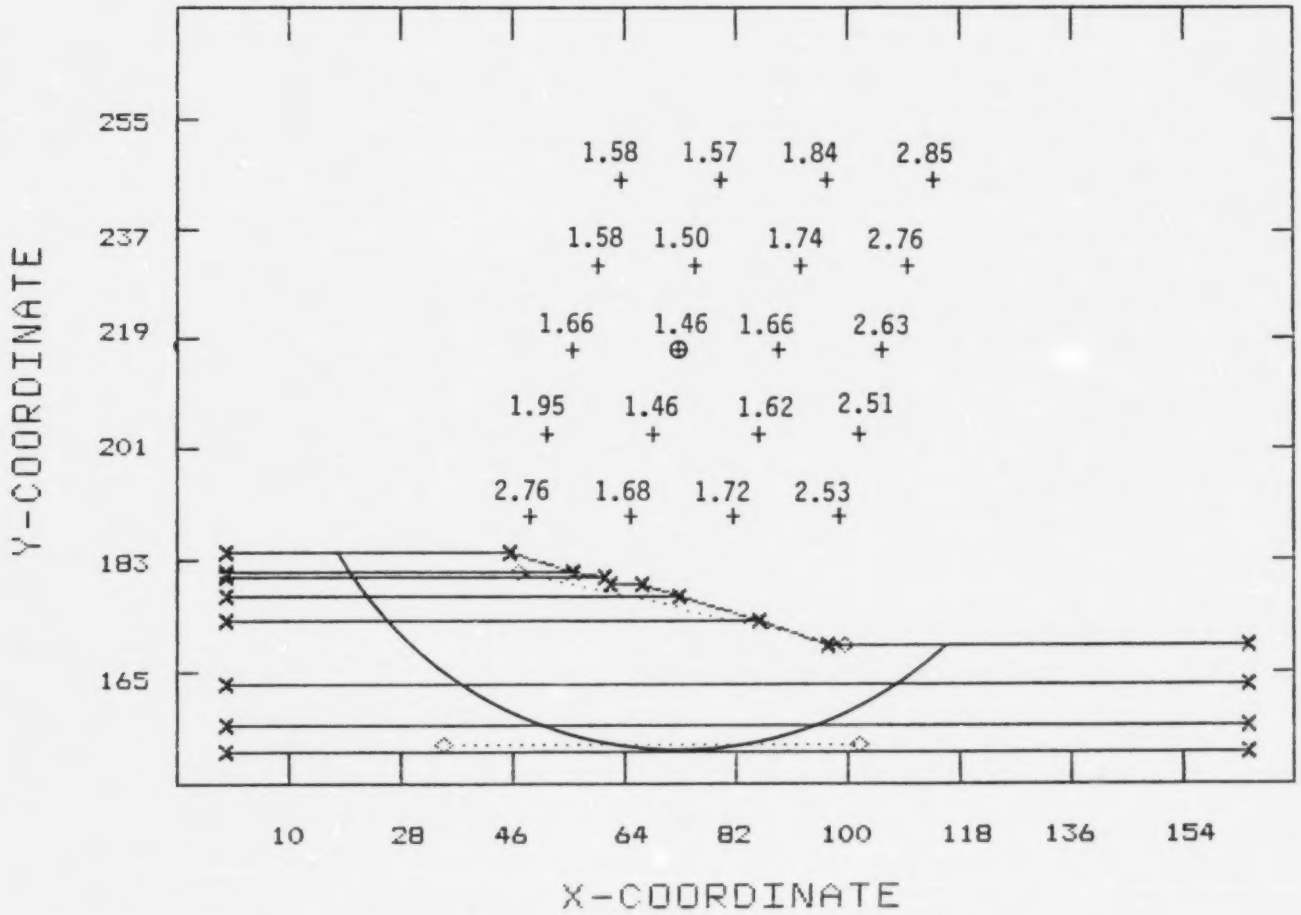
(Case A-2)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH. 85-GT-5

27 OCT 27, 1986

TS.



UNIT WEIGHT	COHESION	PHI	PHI B
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20.3400	130.0000	.0000	.0000
21.2200	95.0000	.0000	.0000
20.6000	65.0000	.0000	.0000
19.2600	61.0000	.0000	.0000
18.6200	61.0000	.0000	.0000
-1.0000	.0000	.0000	.0000

(Case A-4)

APPENDIX A2

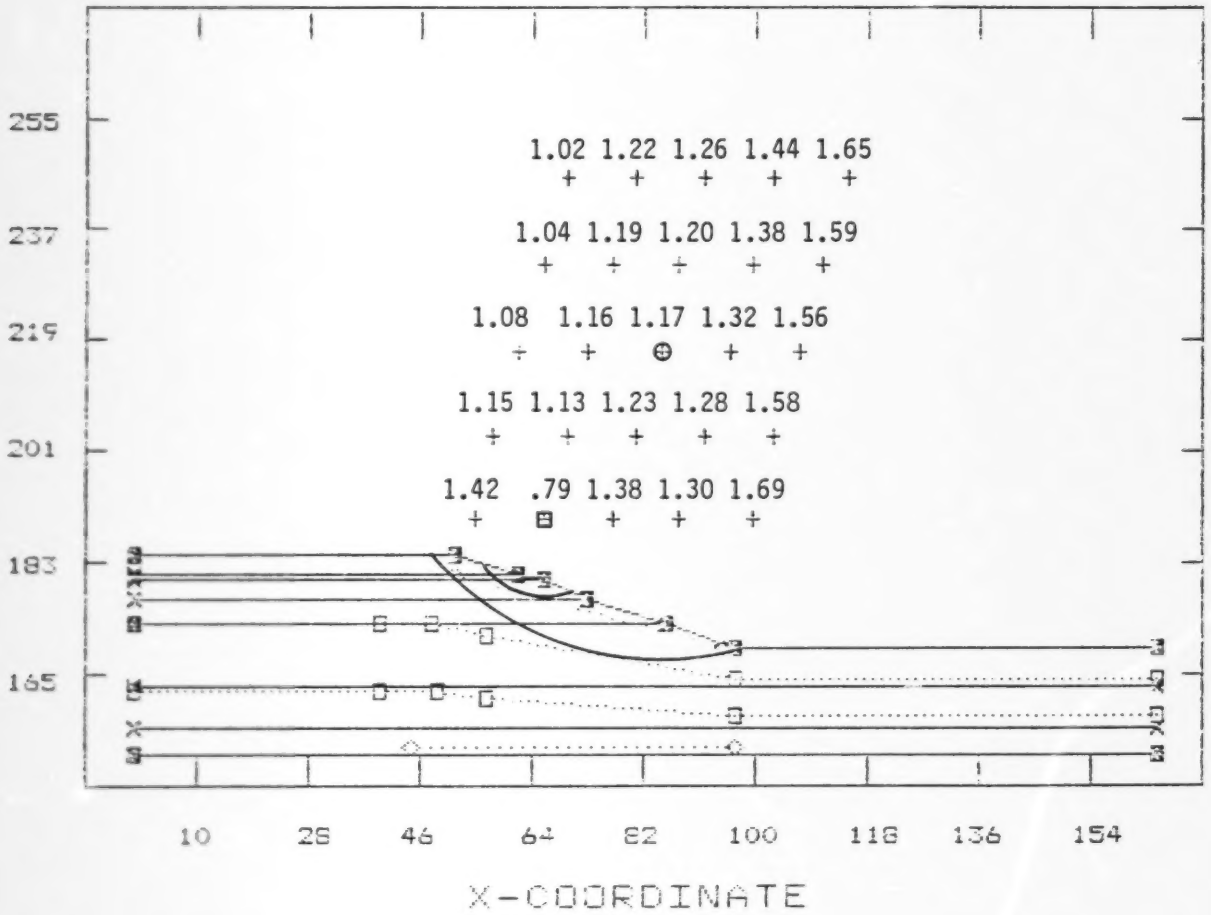
EFFECTIVE STRESS ANALYSIS PLOTS

CROSS-SECTION OF GEOMETRY

CWMC, PHASE 4B GEOTECH, 85-GT-5

1 JULY 17, 1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
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20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

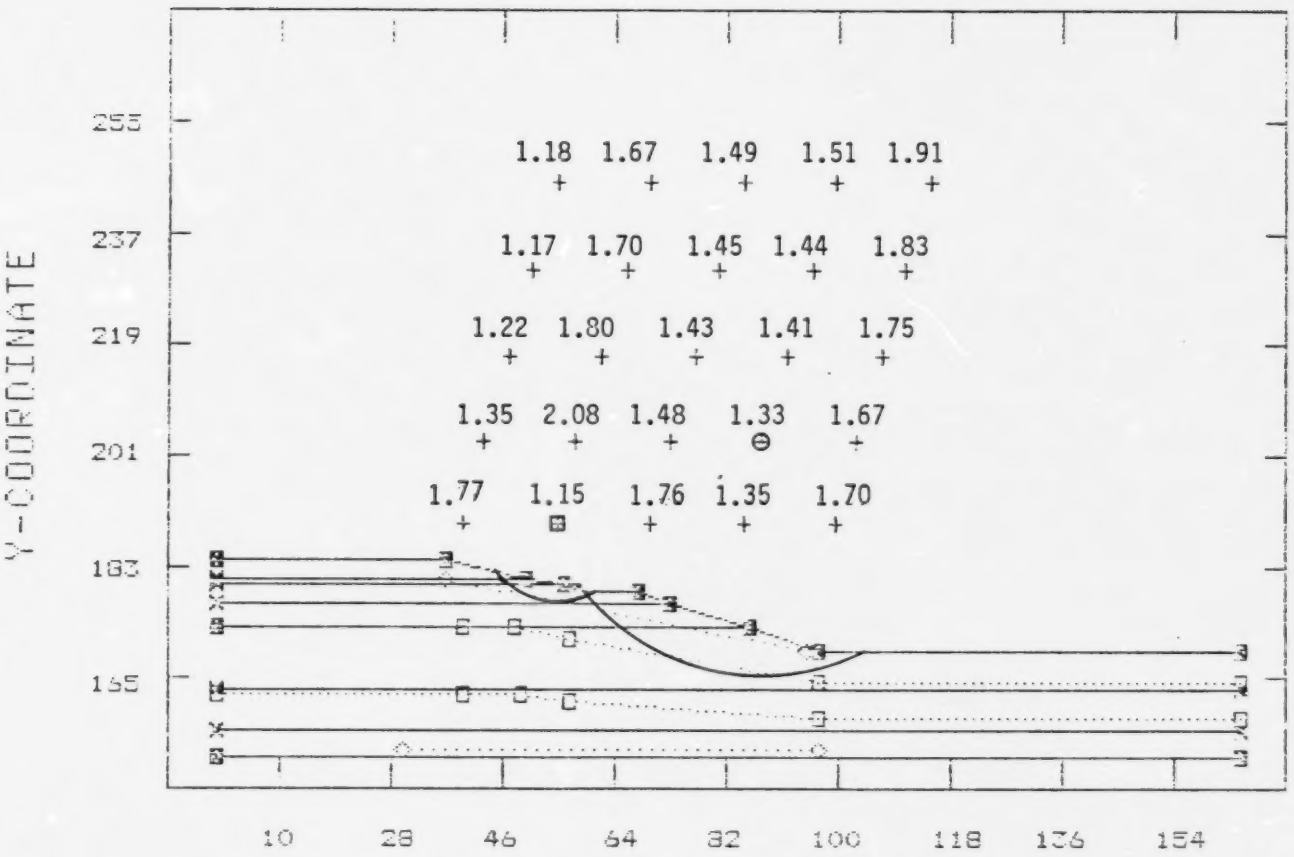
(Case A-1)

CROSS-SECTION OF GEOMETRY

CWMC PHASE 4B GEOTECH. 85-3T-5

2 JULY 17, 1986

SS.



UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

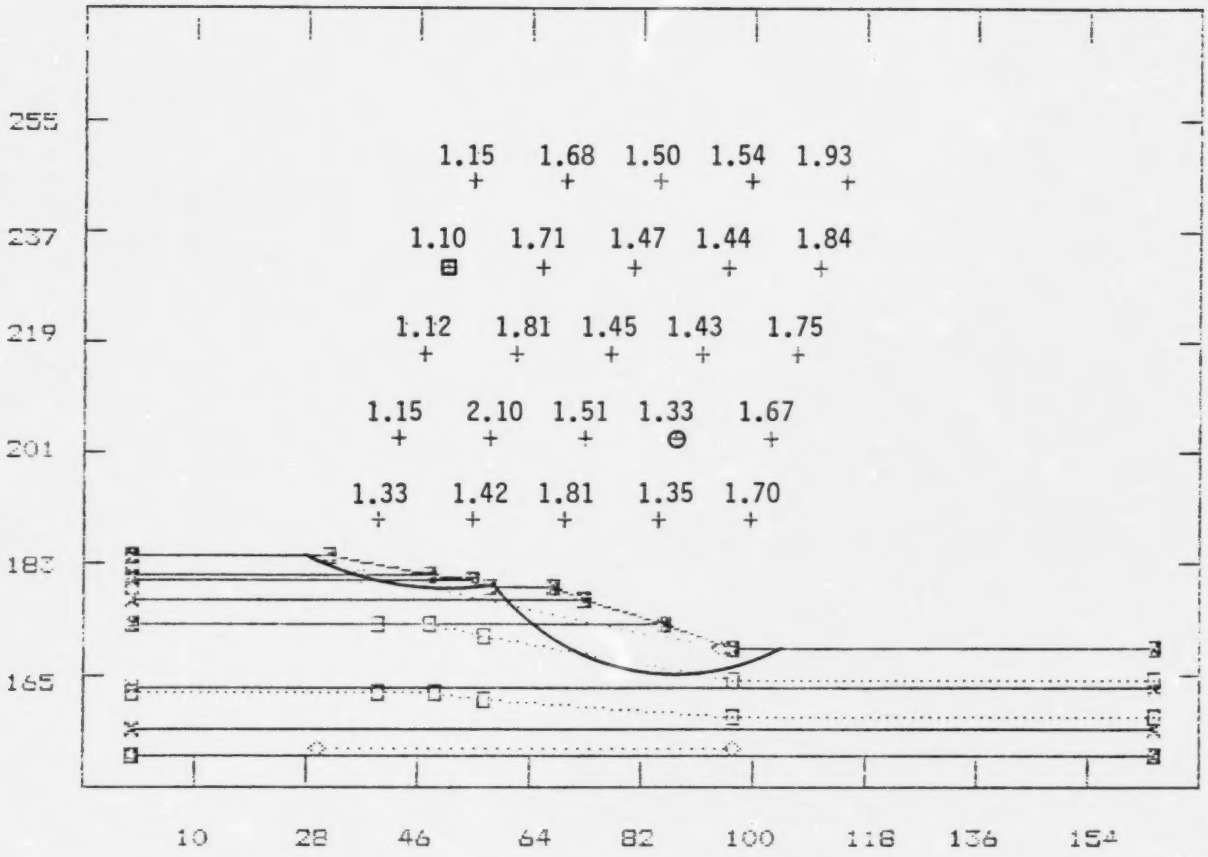
(Case A-6)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

3 JULY 17,1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

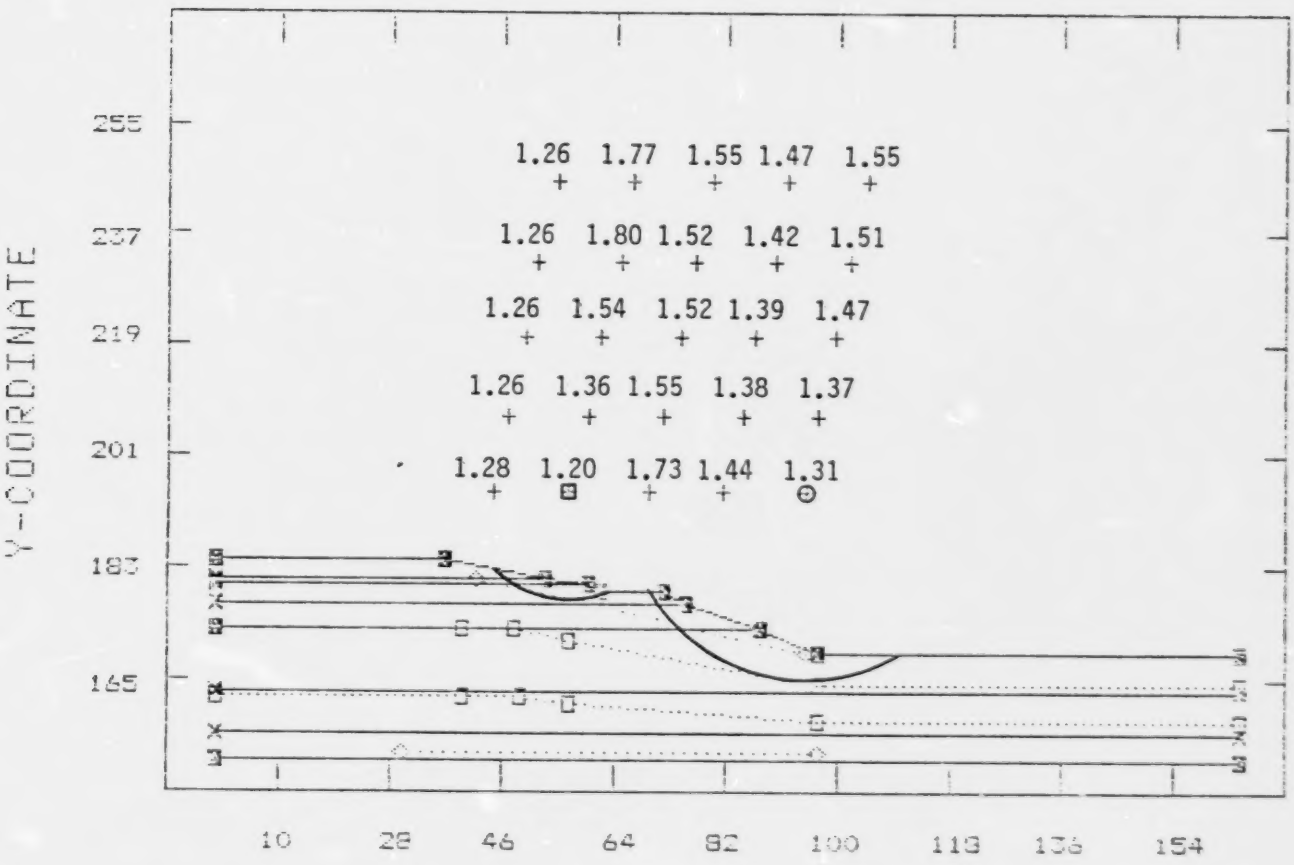
(Case A-14)

CROSS-SECTION OF GEOMETRY

DWMC PHASE 4B GEOTECH. BS-GT-5

4 JULY 17, 1986

ES.



UNIT WEIGHT	COHESION	PHI	PHI B
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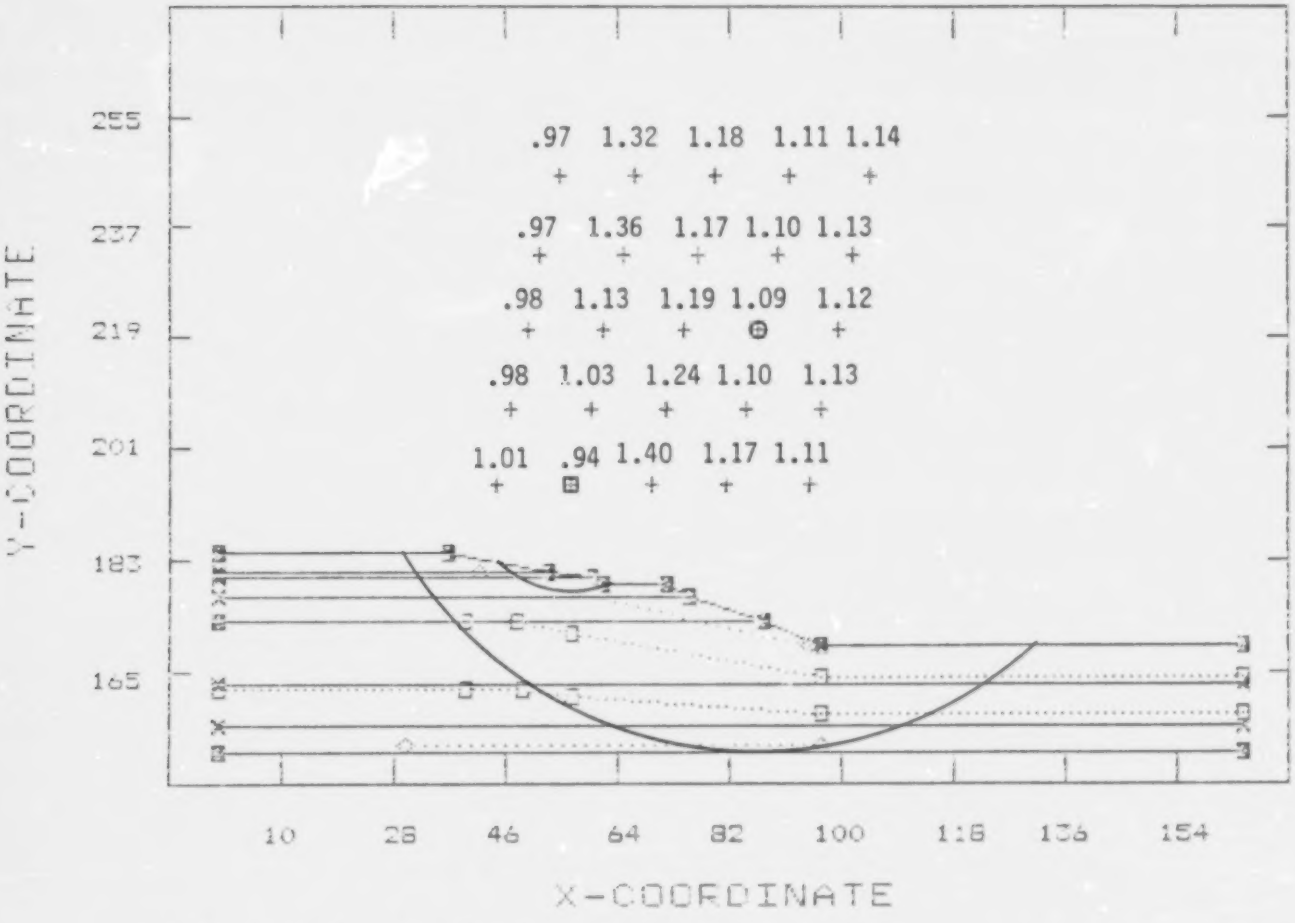
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

(Case A-12)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH. SE-ST-3

JULY 17, 1986



UNIT WEIGHT	COHESION	PHI	PHI B
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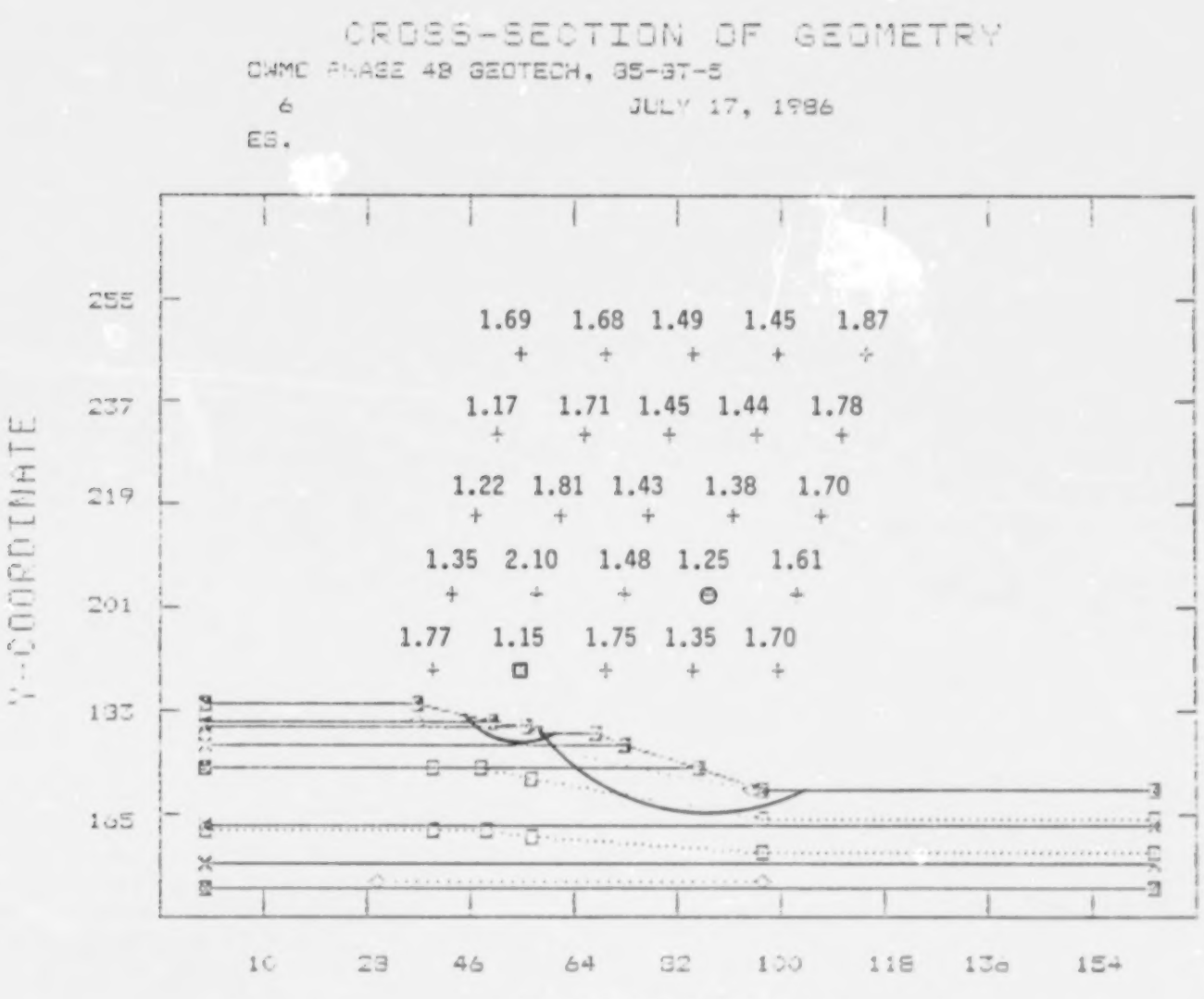
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

(Case A-13)



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

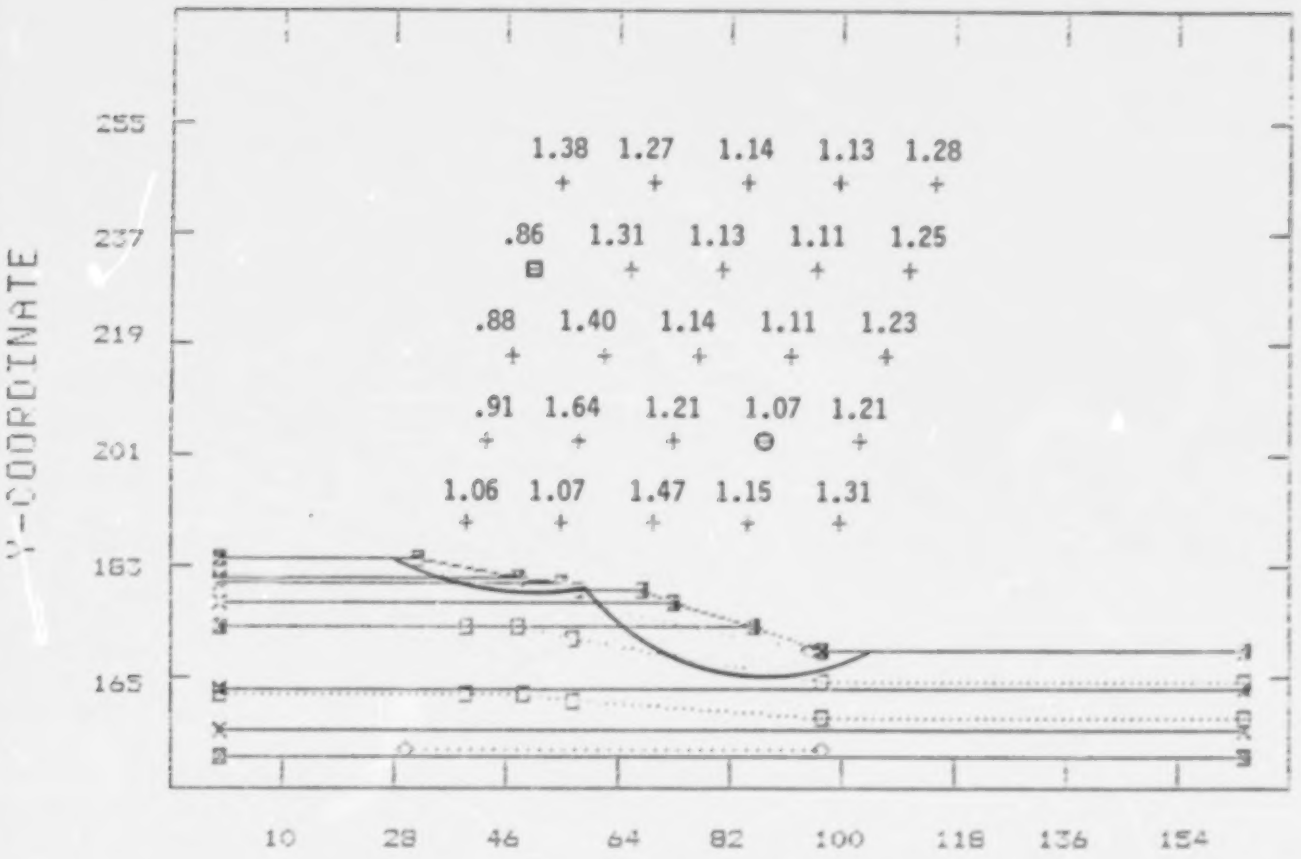
(Case A-7)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-3

7 JULY 17, 1986

ES.



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

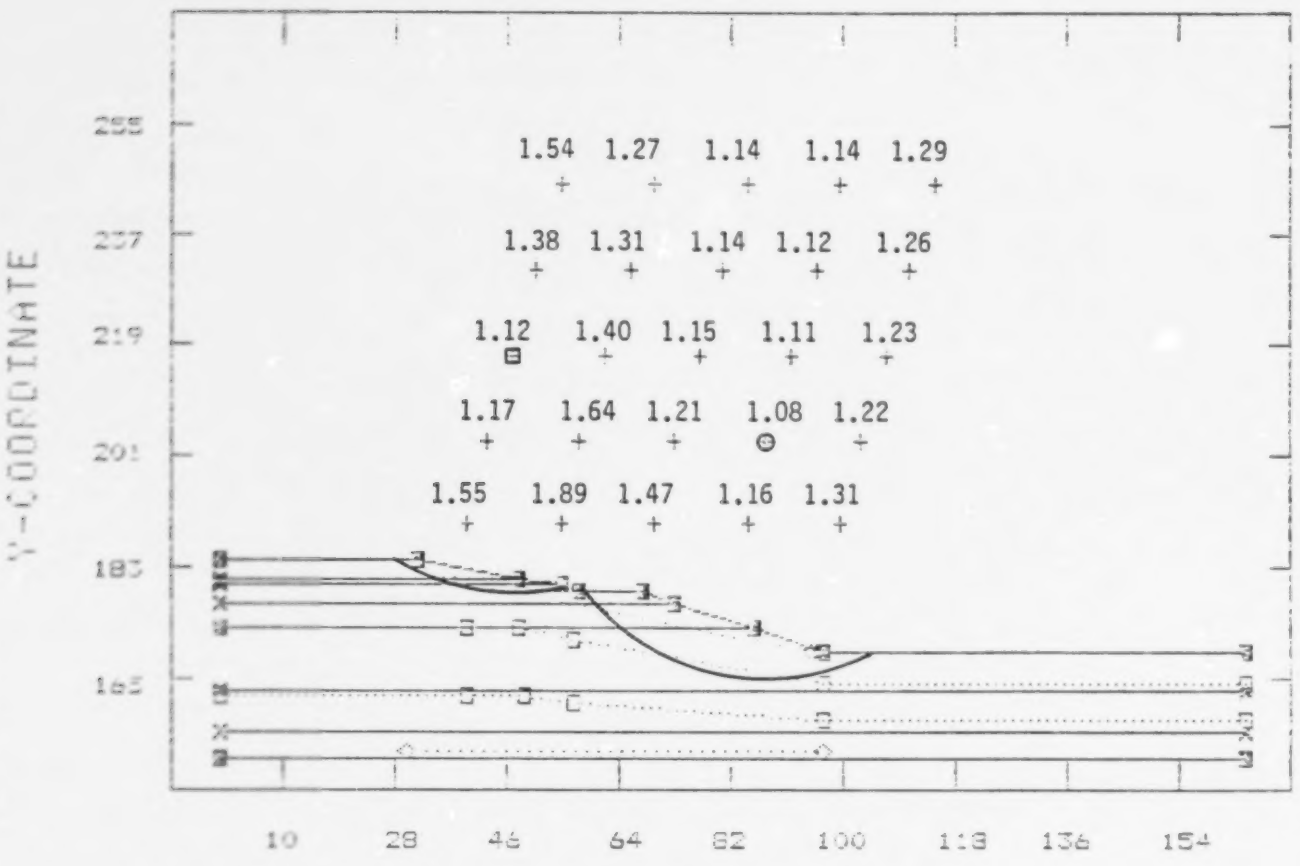
(Case A-15)

CROSS-SECTION OF GEOMETRY

CWMC PHASE 4B GEOTECH. 85-BT-3

B JULY 17, 1986

EB,



UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	.0000	20.0000	.0000
20.3400	5.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
17.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

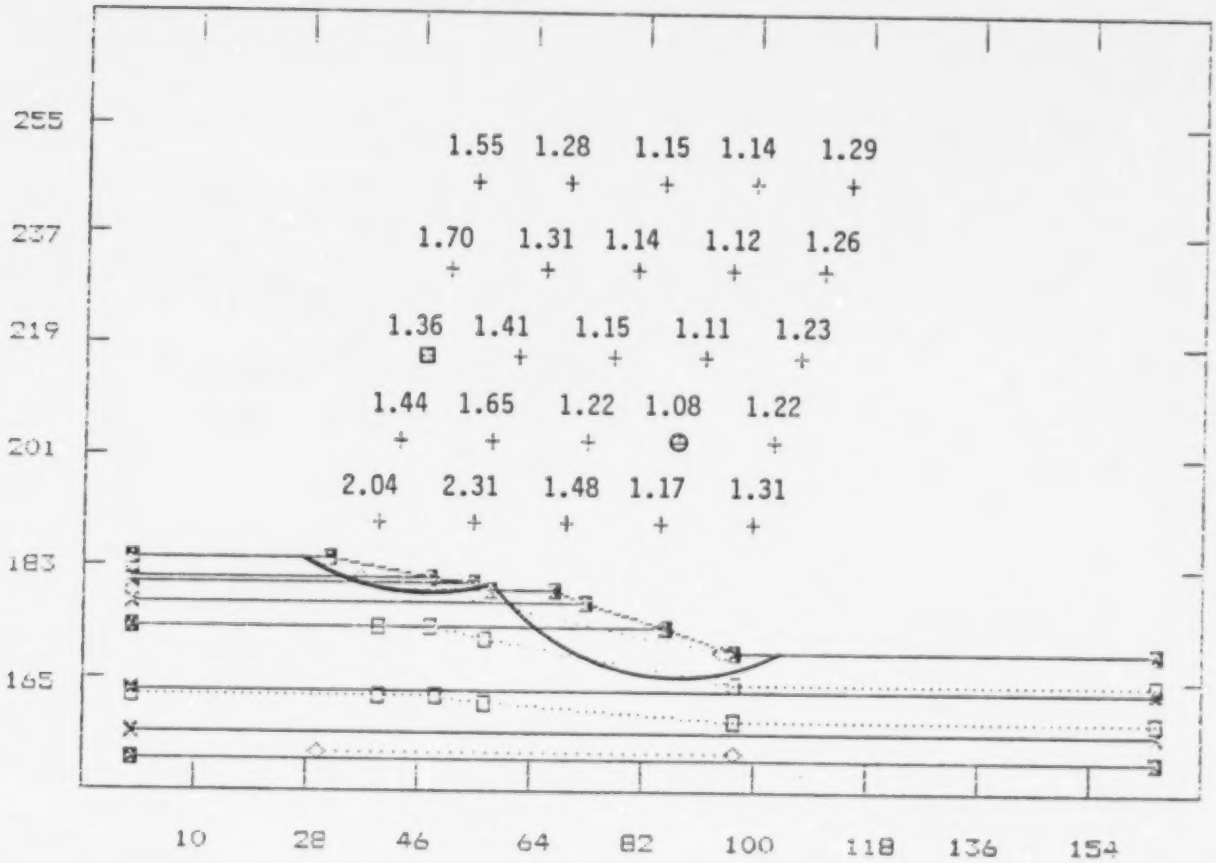
(Case C-1)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, GE-GT-5

9 JULY 17, 1986

E3,



UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	.0000	20.0000	.0000
20.3400	10.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

(Case D-1)

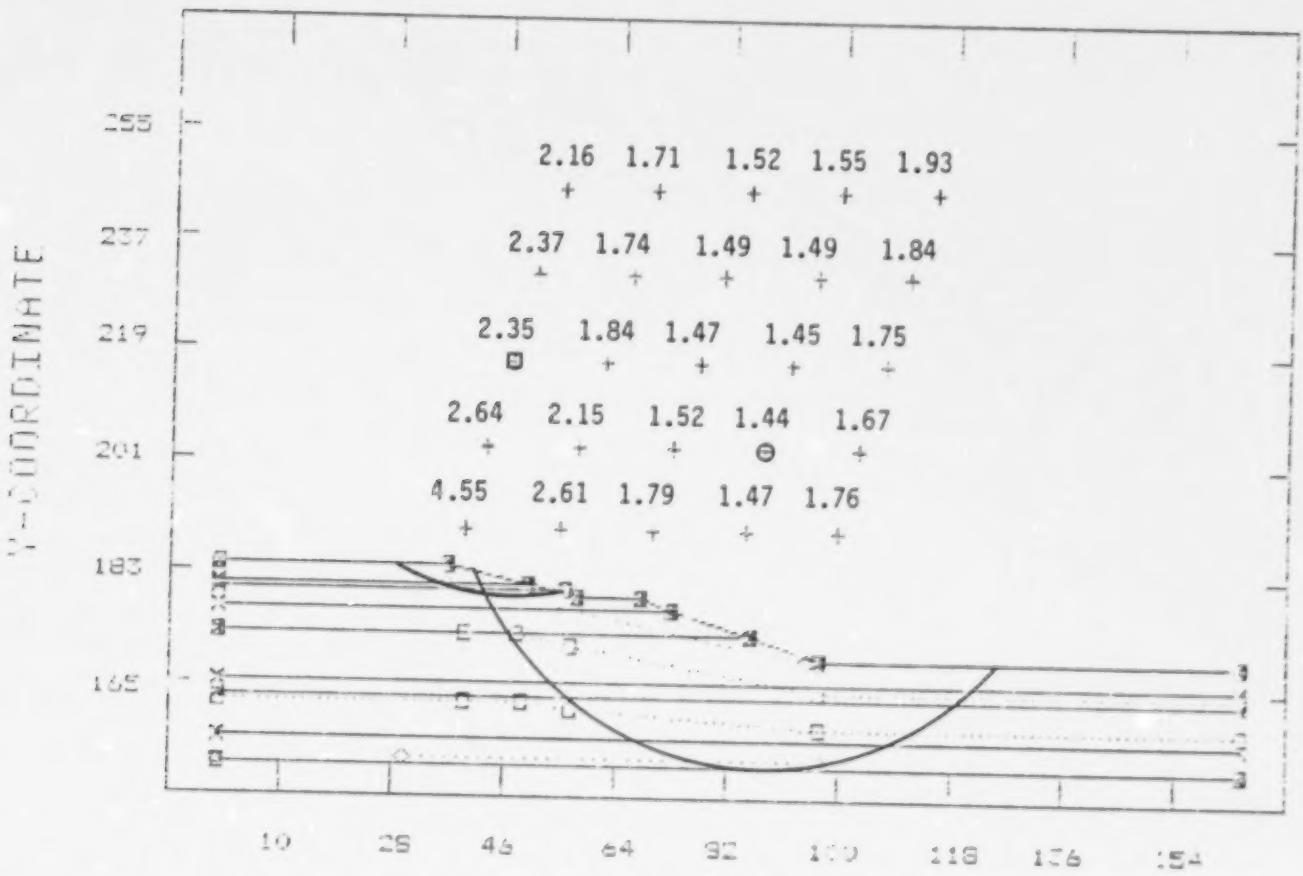
CROSS-SECTION OF GEOMETRY

DWMC PHASE 4B GEOTECH. 85-87-S

1:

JULY 21, 1986

ES.



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	20.0000	30.0000	.0000
19.0500	.0000	19.0000	.0000
20.3400	8.0000	20.0000	.0000
21.2200	20.0000	25.0000	.0000
20.6000	20.0000	20.0000	.0000
20.6000	7.0000	29.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

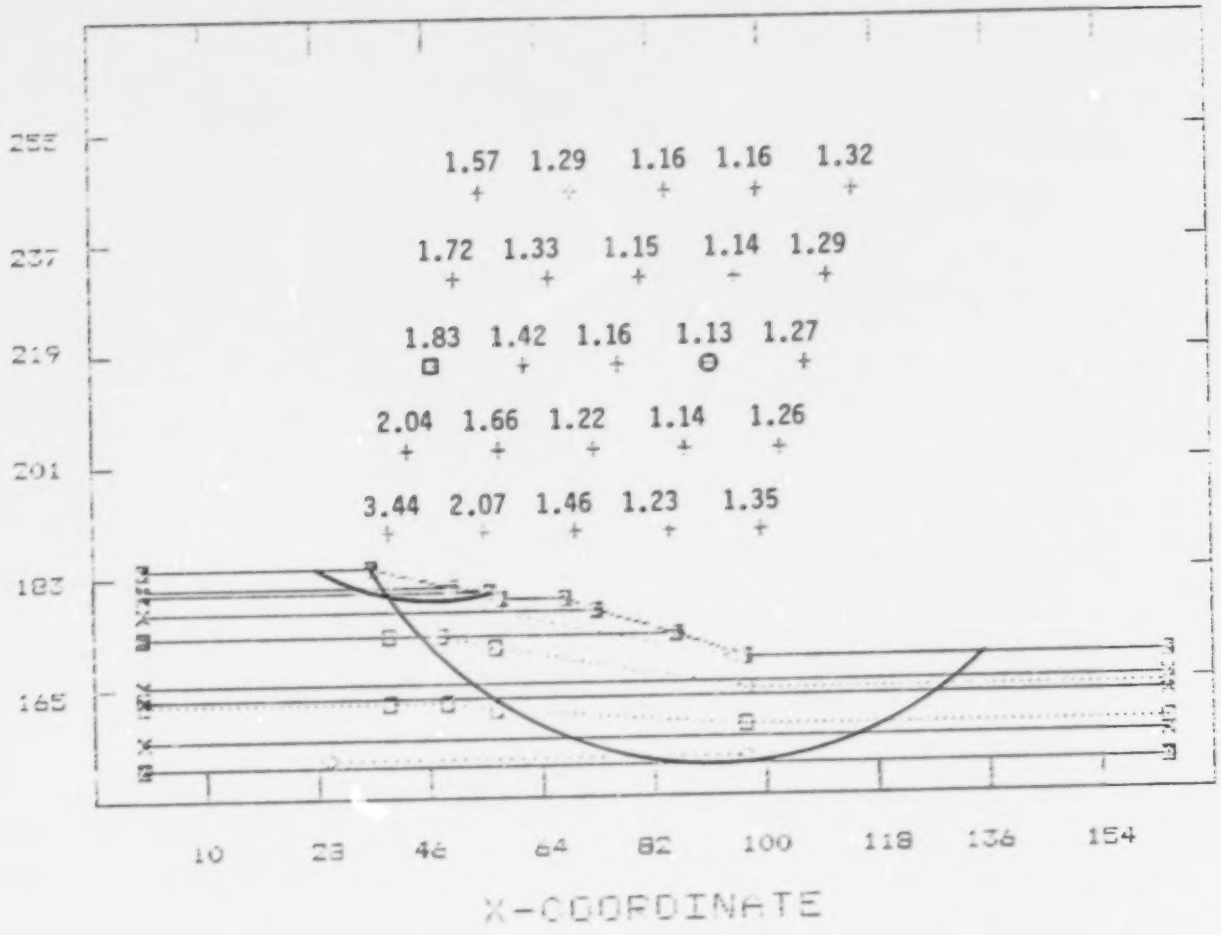
(Case B-2)

CROSS-SECTION OF GEOMETRY

CANC PHASE 4B GEOTECH. GS-3'-E

12 JULY 21,1988

EB.



UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	.0000	19.0000	.0000
20.3400	8.0000	20.0000	.0000
21.2200	20.0000	25.0000	.0000
20.6000	20.0000	20.0000	.0000
20.5000	7.0000	29.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

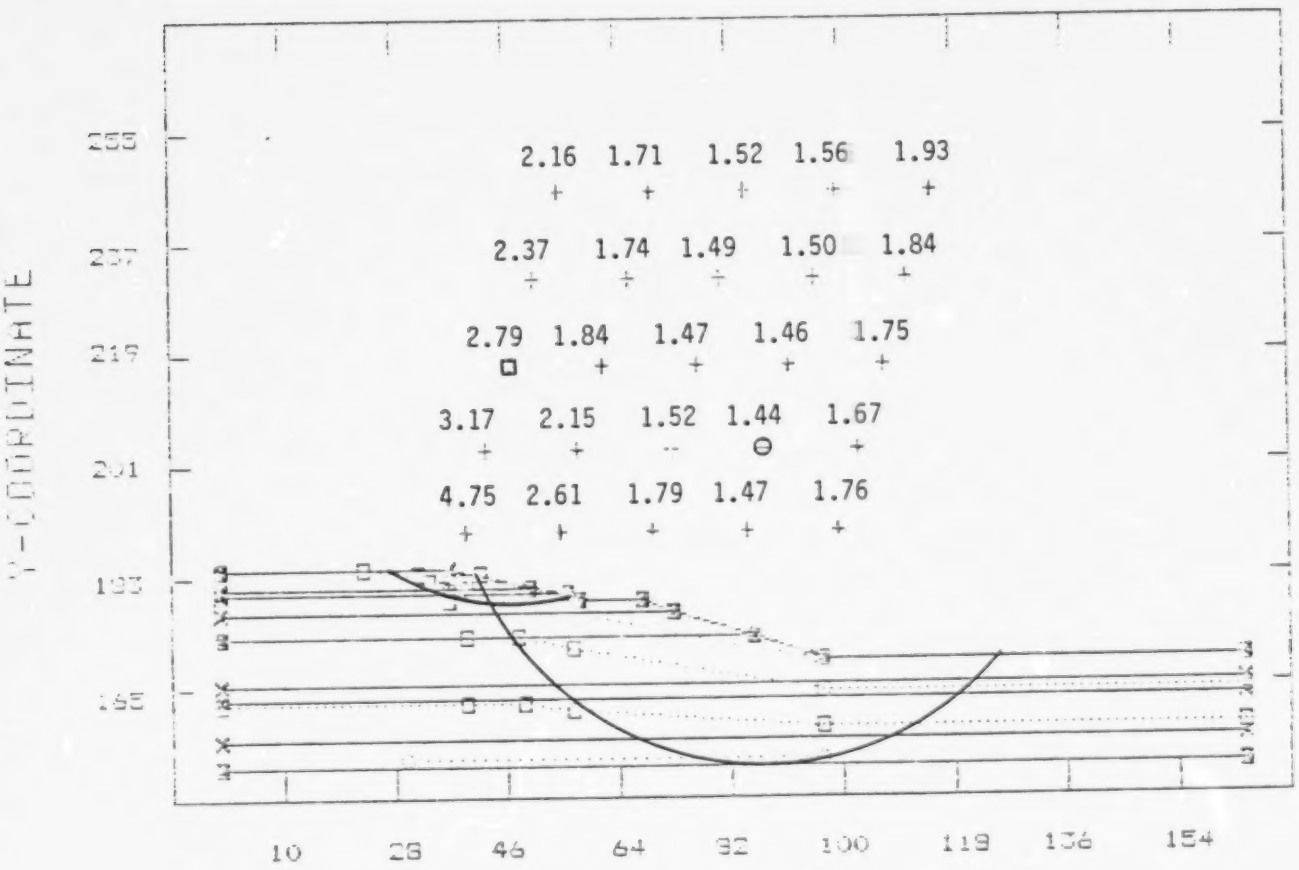
(Case B-6)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH. BE-BT-E

13 JULY 21,1986

EB.



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	20.0000	30.0000	.0000
19.0500	.0000	19.0000	.0000
20.3400	8.0000	20.0000	.0000
21.2200	20.0000	25.0000	.0000
20.6000	20.0000	20.0000	.0000
20.6000	7.0000	29.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

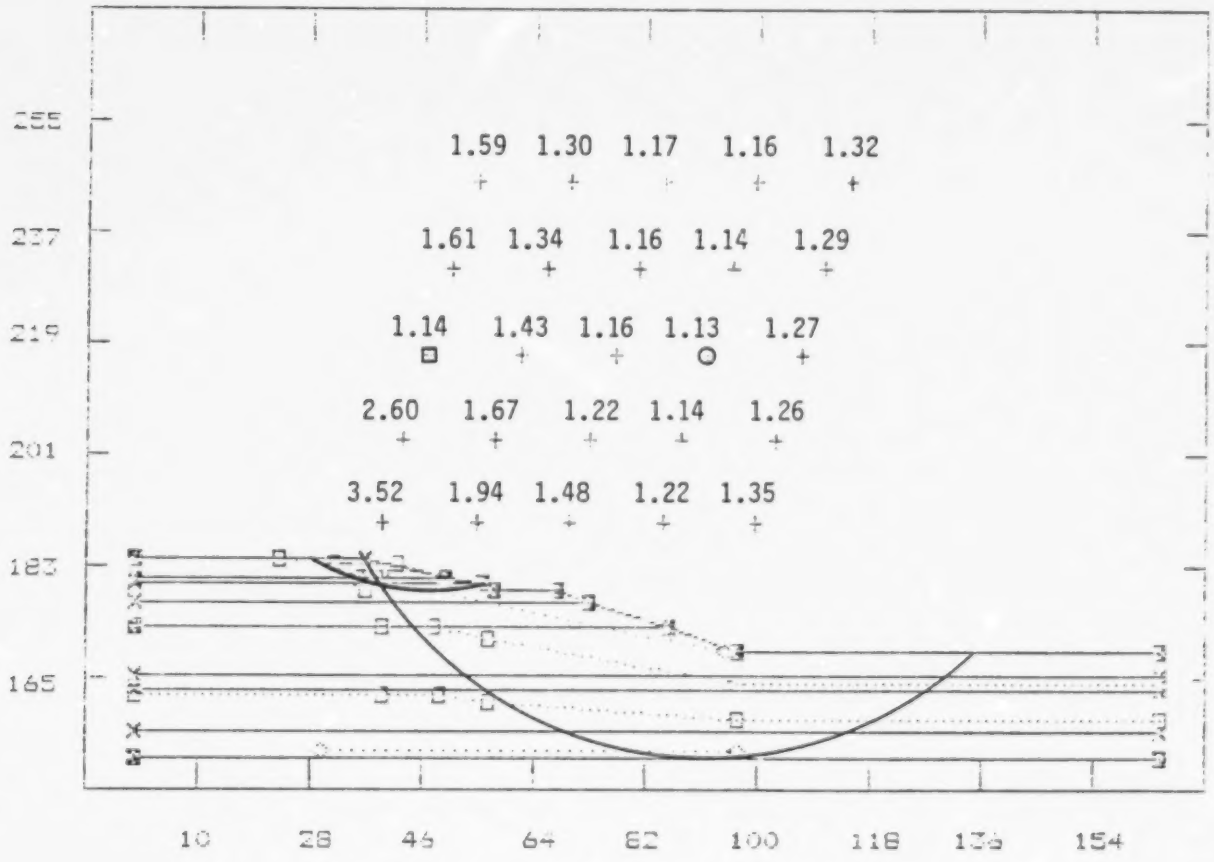
(Case B-3)

CROSS-SECTION OF GEOMETRY

CAMC PHASE 4B GEOTECH, 85-ST-5

14 JULY 21,1986

85,



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	20.0000	30.0000	.0000
19.0500	.0000	19.0000	.0000
20.3400	8.0000	20.0000	.0000
21.2200	20.0000	25.0000	.0000
20.6000	20.0000	20.0000	.0000
20.6000	7.0000	29.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

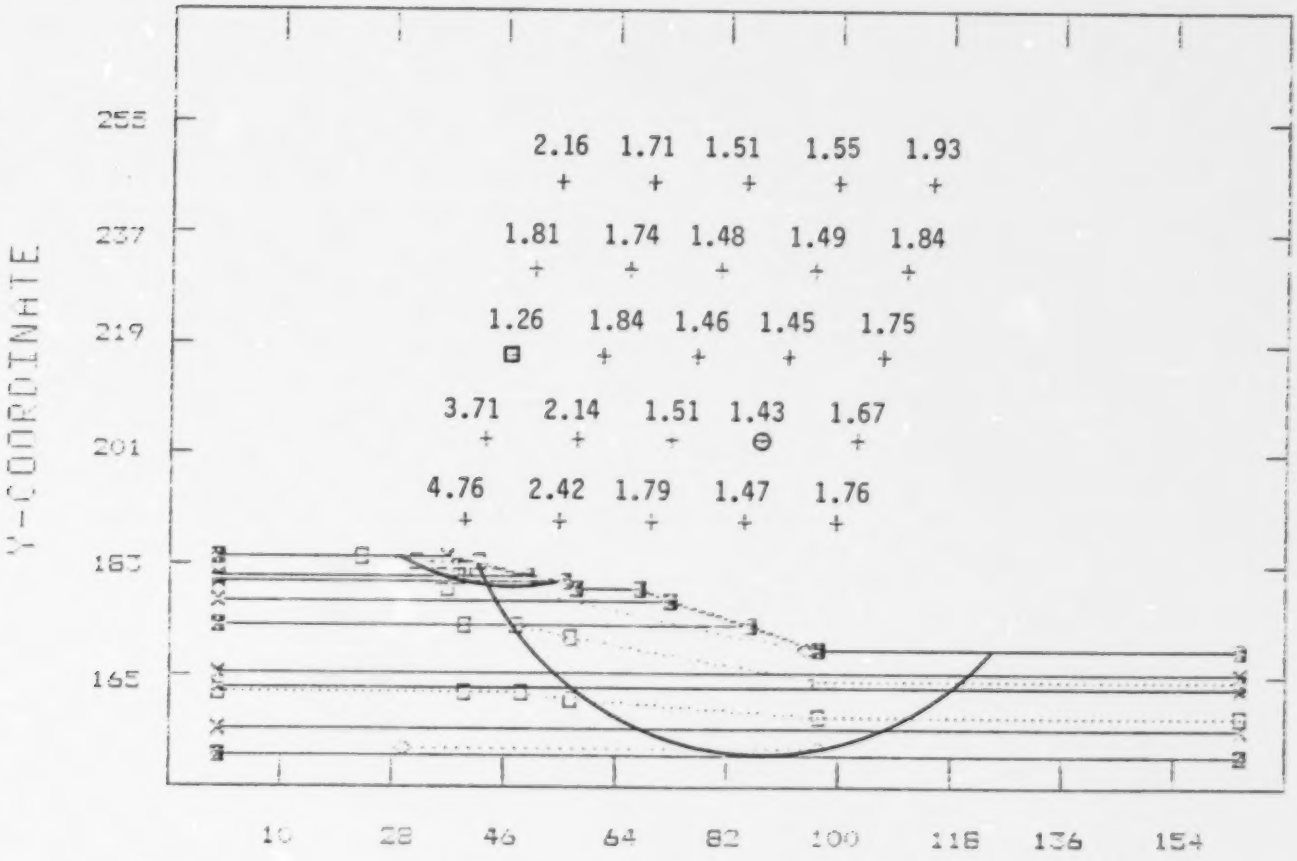
(Case B-4)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

15 JULY 21,1986

ES.



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	20.0000	30.0000	.0000
19.0500	.0000	19.0000	.0000
20.3400	8.0000	20.0000	.0000
21.2200	20.0000	25.0000	.0000
20.6000	20.0000	20.0000	.0000
20.6000	7.0000	29.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

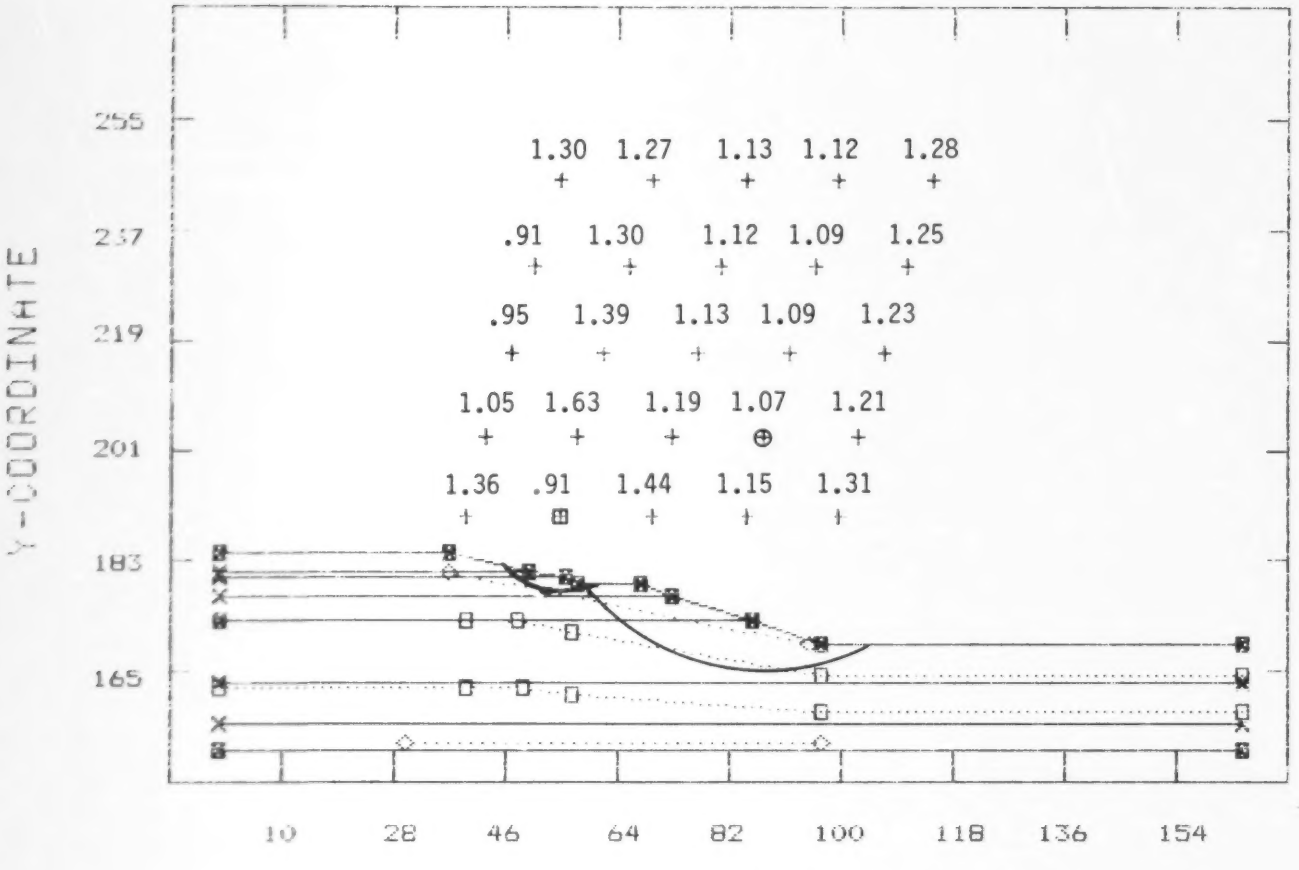
(Case B-5)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

16 JULY 27, 1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

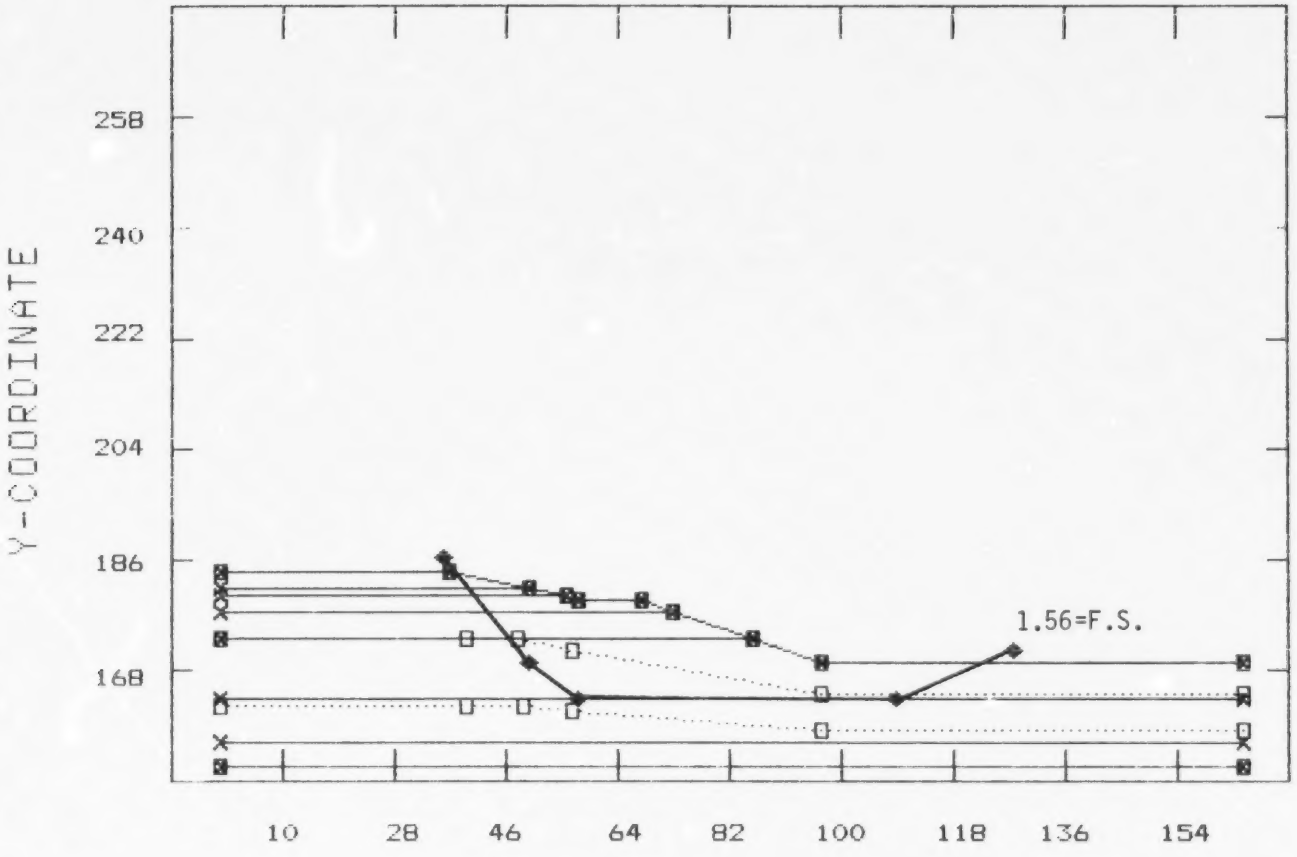
(Case A-8)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

17 JULY 27, 1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

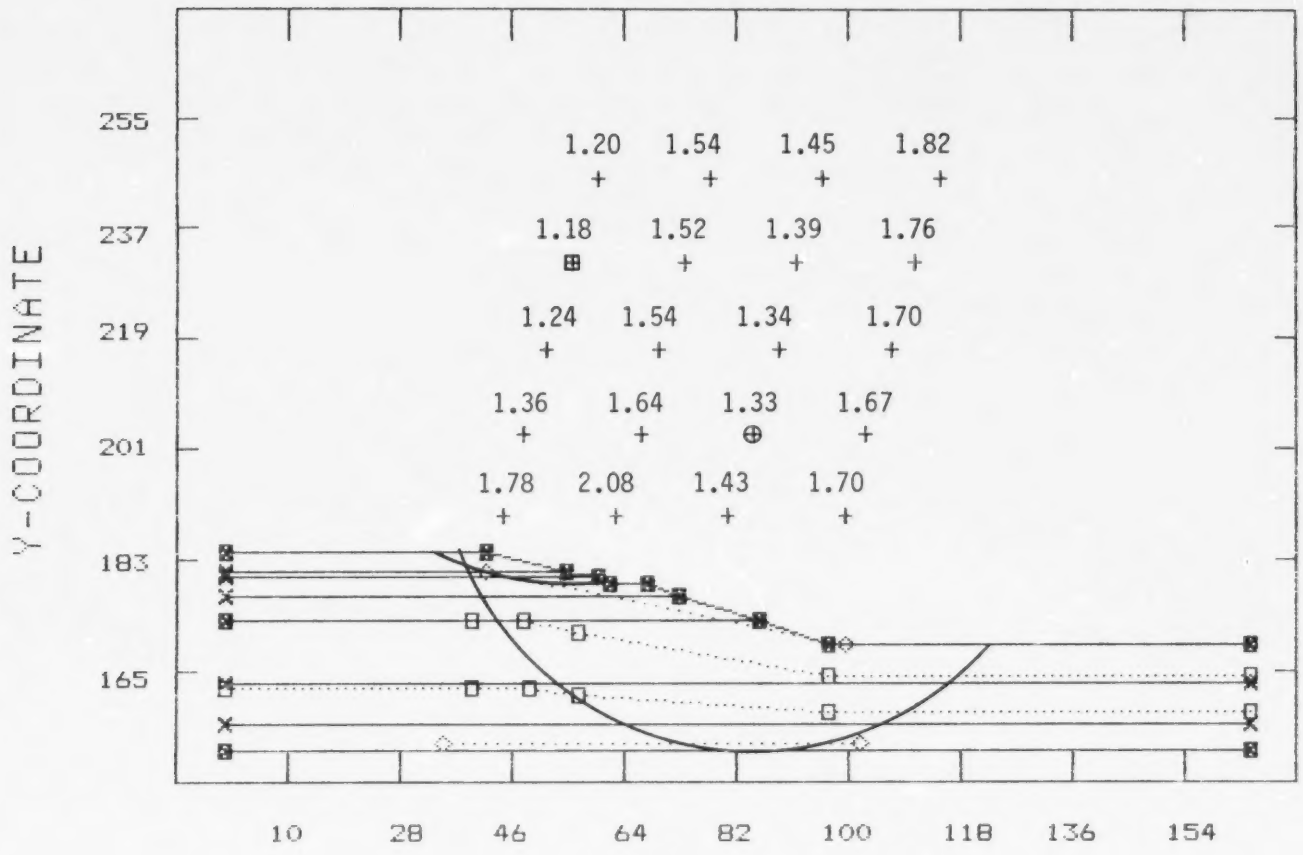
(Case A-9)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH,85-GT-5

18 OCT 27, 1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

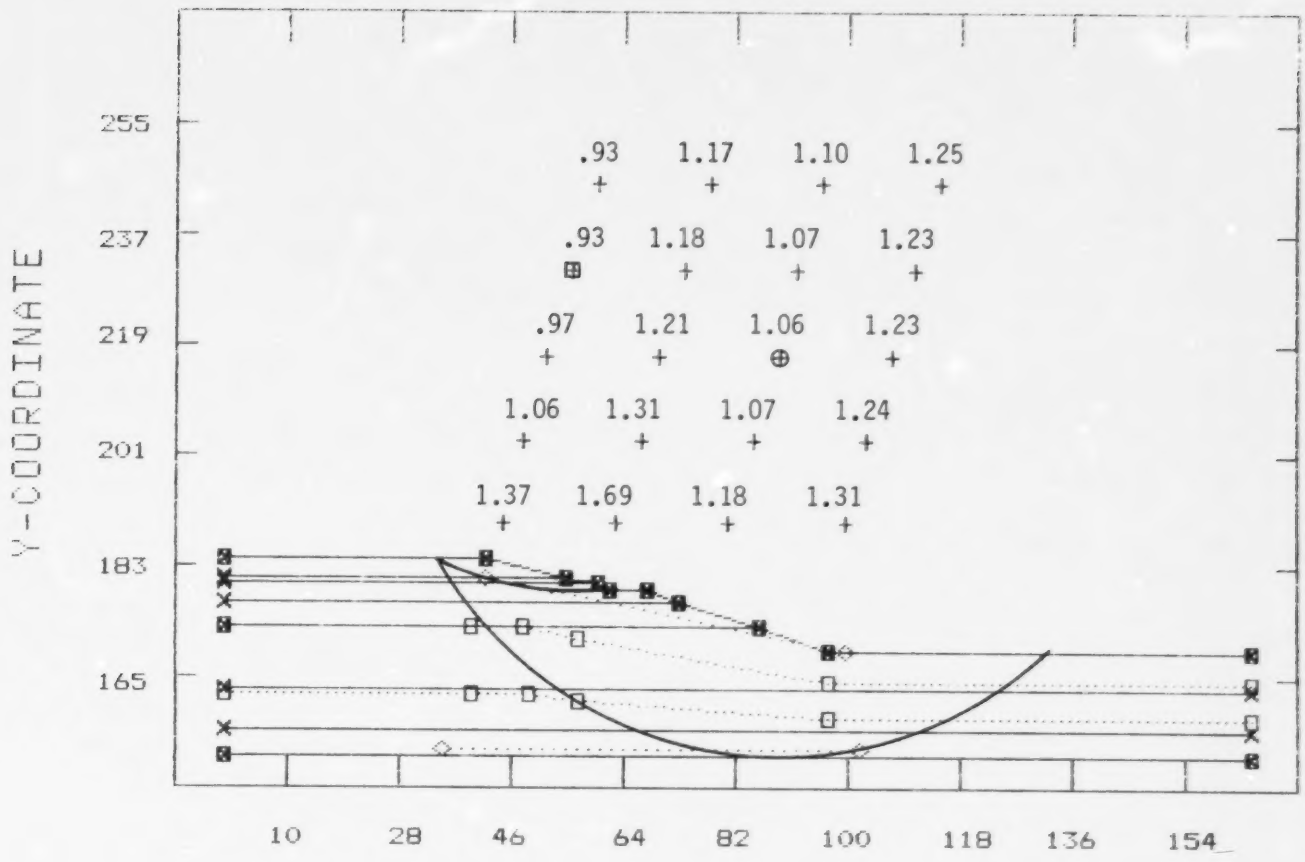
(Case A-10)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

19 OCT 27, 1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

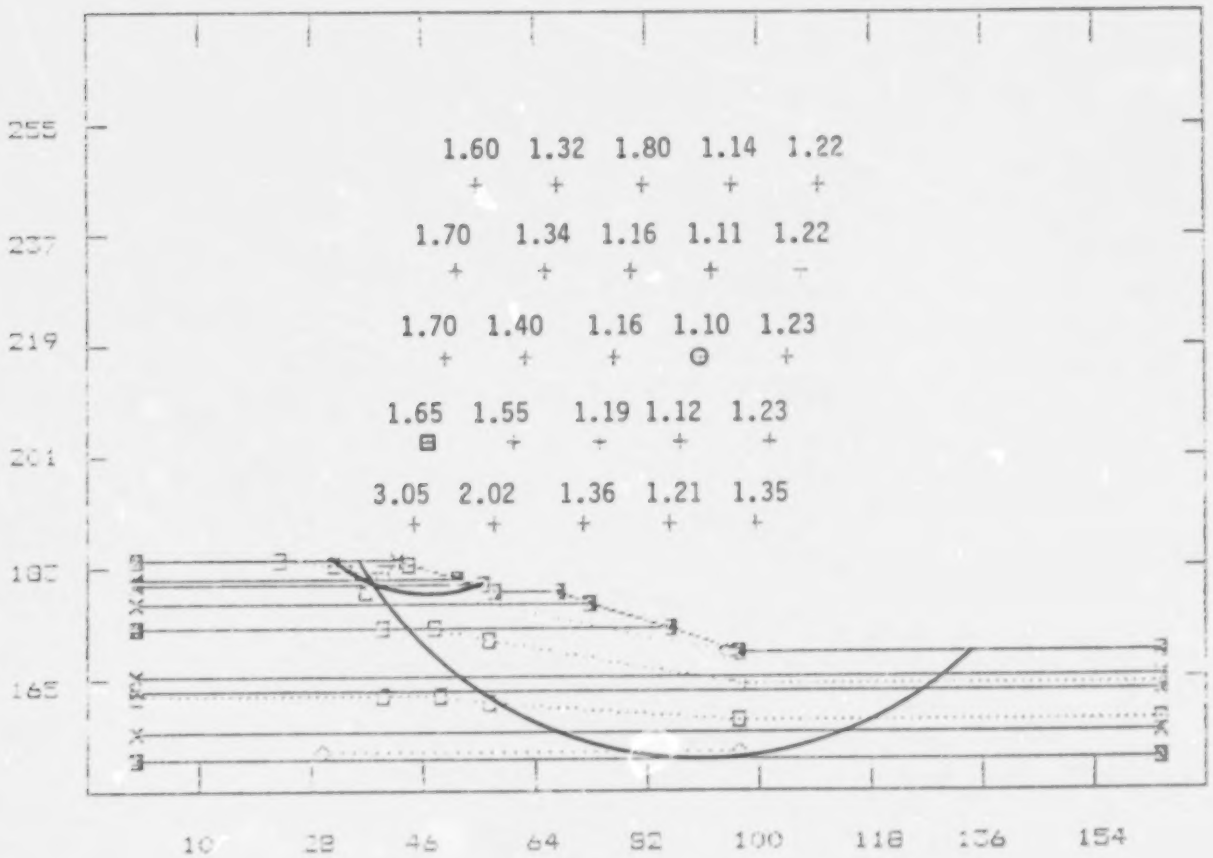
(Case A-11)

CROSS-SECTION OF GEOMETRY

DWMC PHASE 4B GEOTECH. 85-01-E

20 JULY 21,1986

ES.



UNIT WEIGHT	COHESION	PHI	PHI B
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19.0500	.0000	19.0000	.0000
20.3400	8.0000	20.0000	.0000
21.2200	20.0000	25.0000	.0000
20.6000	20.0000	20.0000	.0000
20.6000	7.0000	29.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

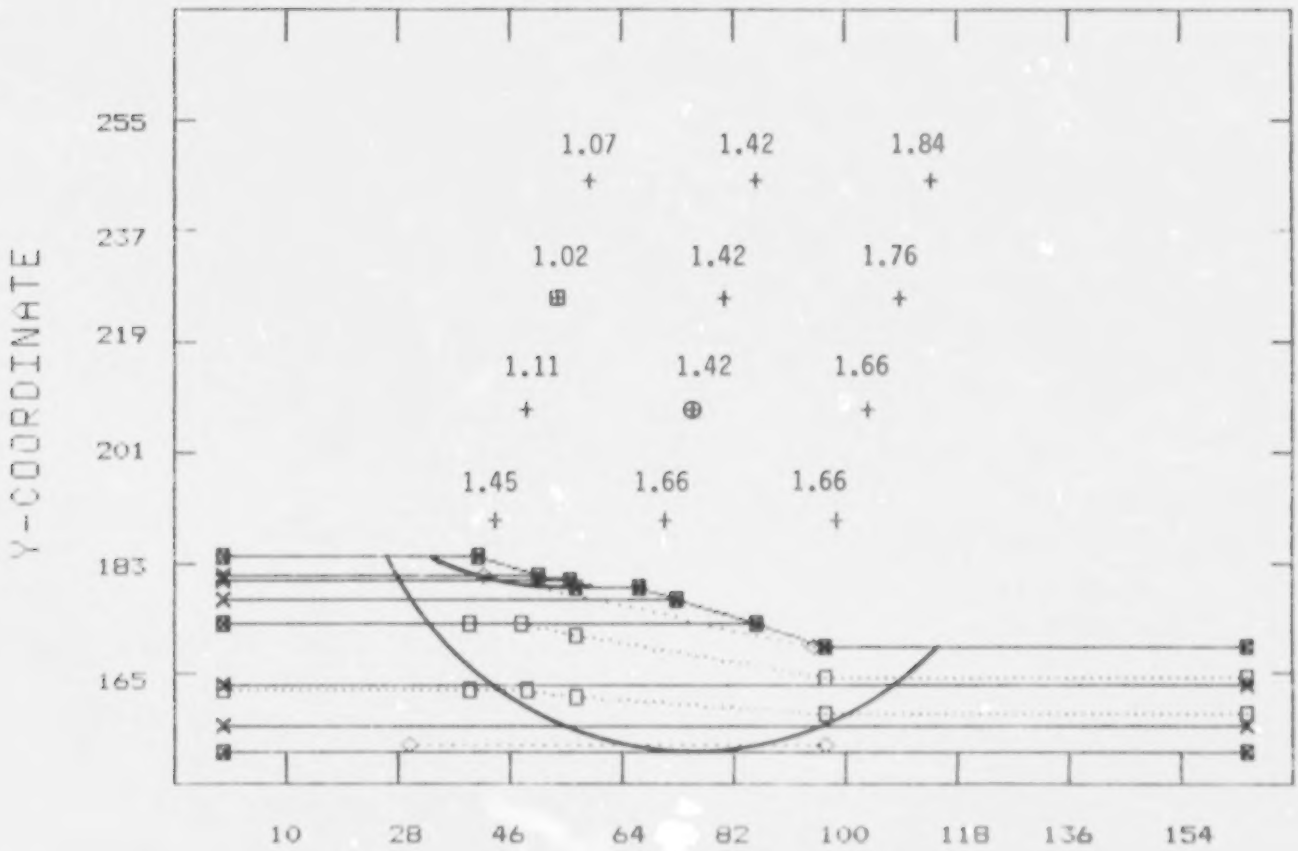
(Case B-1)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

22 OCT 27, 1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

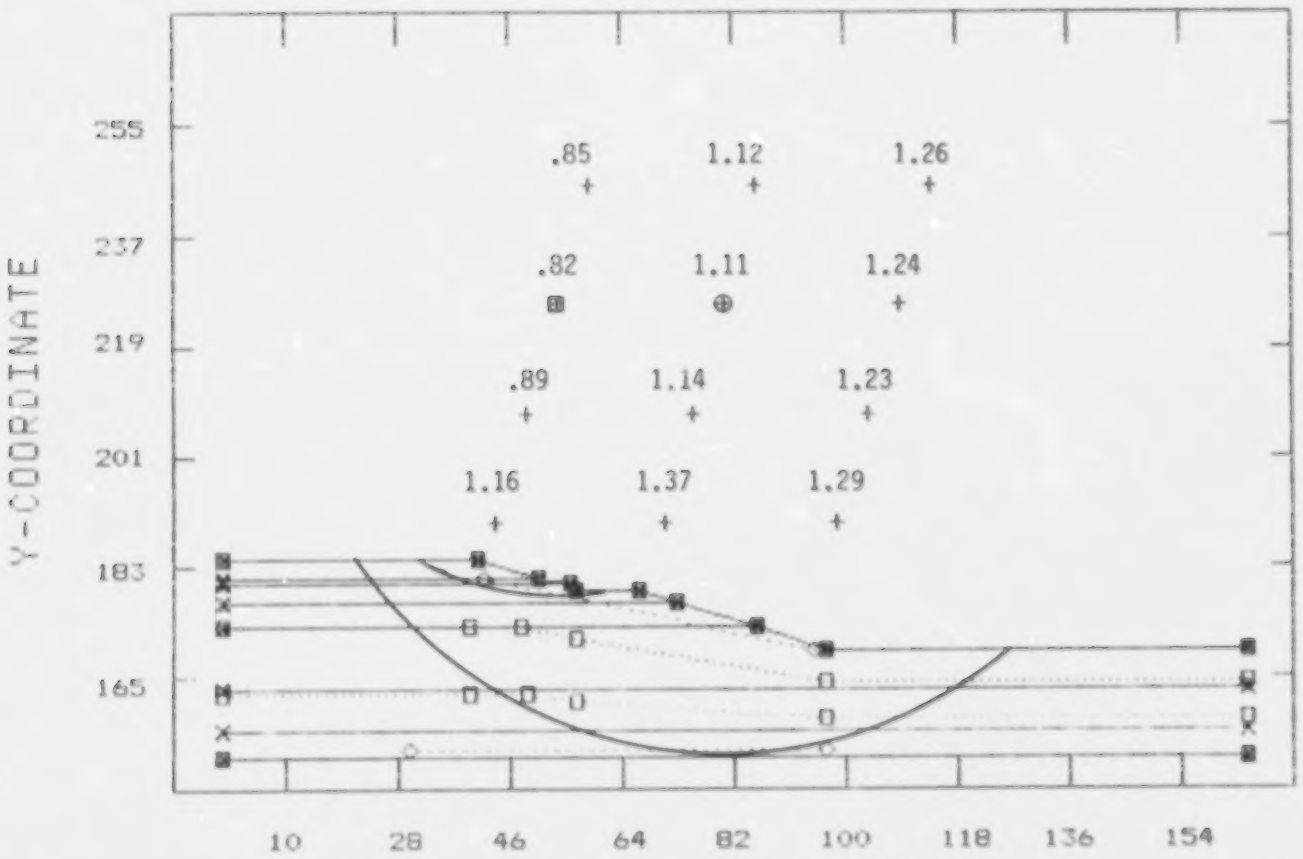
(Case A-2)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH. 85-GT-5

23 OCT 27, 1986

ES.



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

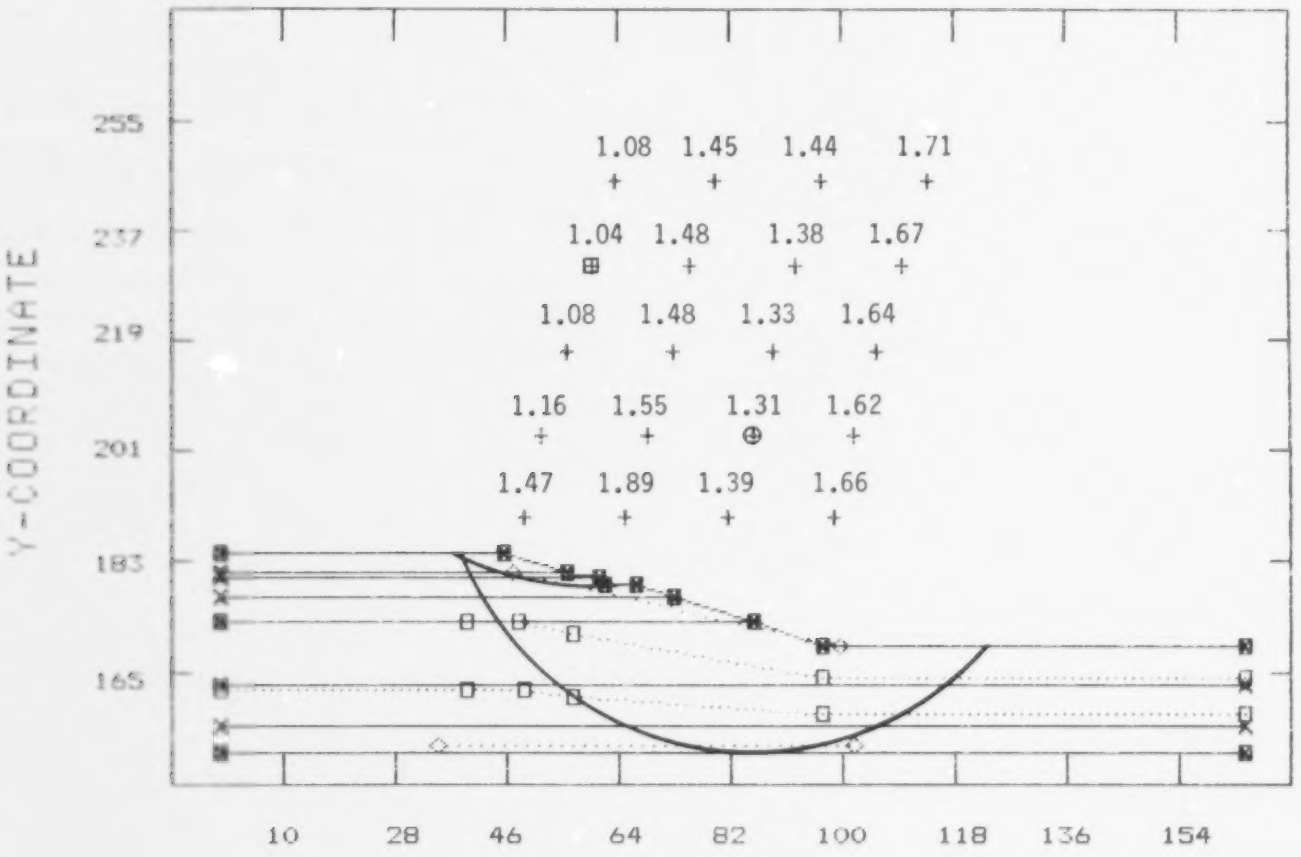
(Case A-3)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH. 85-GT-5

25 OCT 27, 1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

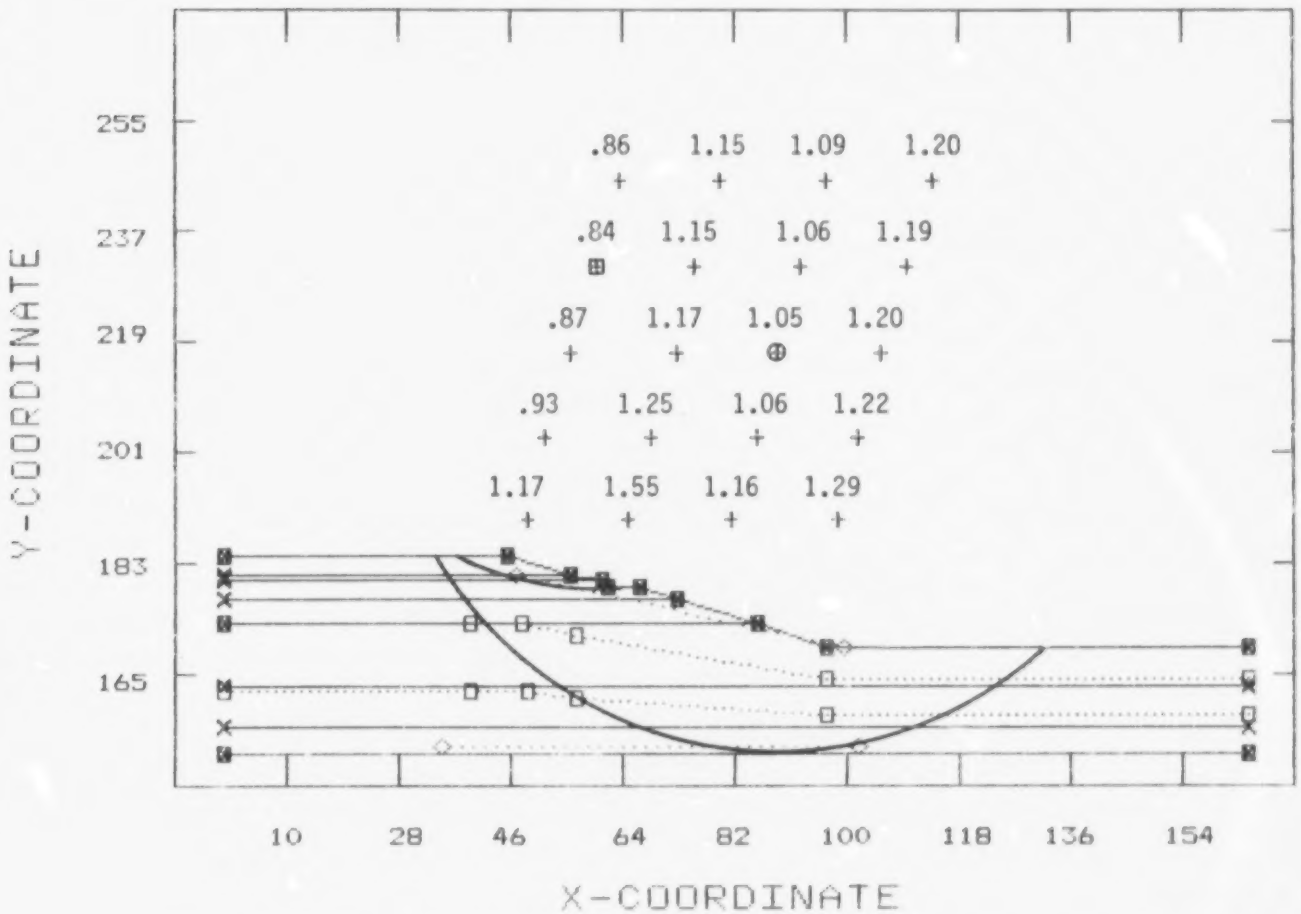
(Case A-4)

CROSS-SECTION OF GEOMETRY

OWMC PHASE 4B GEOTECH, 85-GT-5

26 OCT 27, 1986

ES,



UNIT WEIGHT	COHESION	PHI	PHI B
20.3800	.0000	20.0000	.0000
19.0500	.0000	20.0000	.0000
20.3400	.0000	20.0000	.0000
21.2200	10.0000	25.0000	.0000
20.6000	10.0000	25.0000	.0000
19.2600	5.0000	23.0000	.0000
18.6200	.0000	17.0000	.0000
-1.0000	.0000	.0000	.0000

(Case A-5)

APPENDIX B

PC-SLOPE MODEL

APPENDIX B

B1 DESCRIPTION

Author:	Geo-Slope Programming Ltd.
Computer System:	IBM AT
References:	Fredlund, 1985
Model Type:	Mathematical, iterative, analytical
Number of Dimensions:	2-D
Assumptions:	• the soil behaves as a Mohr-Coulomb material, • The factor of safety of the cohesive component of strength and frictional component of strength are equal for all soils involved, and • the factor of safety is the same for all slices.
Input Parameter:	• geometry of slope profile • shear strength and unit weight data for each soil unit • pore water pressures • additional loadings to slope • geometry of trial failure circles

Output

- profiles of slope geology
- factor of safety for each trial failure circle
- critical factor of safety for the slope stability analyses.

B2 THEORY

The PC-Slope computer program utilizes the theory of limit equilibrium of forces and moments to compute the factor of safety against movement. The factor of safety is defined as that factor by which the strength parameters must be reduced in order to bring the mass of soil into a state of limiting equilibrium along a selected slip surface.

For an effective stress analysis, the shear strength is defined as:

where:

$$s = c' + (\sigma n - u) \tan \phi'$$

s = shear strength
c' = effective cohesion
ϕ' = effective angle of internal friction
σ_n = total normal stress
u = pore water pressure

For a total stress analysis, the undrained shear strength is used, stresses are defined in terms of total stress and pore-water pressures are not required. (Fredlund, 1985).

Our analyses have been performed using the Bishop's simplified method of slices which is one of a number of limit equilibrium analysis methods available. The analysis technique is based upon the theory presented by Bishop (1955).

The solution used in the program to determine the factor of safety can simply be determined as:

Factor of Safety =

$$\frac{\text{available shear resistance due to cohesion and friction}}{\text{total shearing force on the trial failure surface}}$$

B3 SUMMARY OF INPUT DATA

The profiles and shear strength data used in the various analyses are shown in Appendix B5.

B4 SAMPLE OUTPUT

The following page shows a sample of the output provided by the PC-Slope model.

GEO-SLOPE Ltd. \ P C - S L O P E \ SERIAL NO. 85029
Calgary, Alberta \ Slope Stability Analysis \ GARTNER LEE ASSOCIATES LTD.
\ (C) Copyright, 1985 \

PROJECT NAME ..: OWMC PHASE 4B, GEOTECH 85-GT-5
TRIAL NUMBER .: 1 DATE ..: JUNE 30, 1986
COMMENTS .....: TOTAL STRESS ANALYSIS, 3:1 SLOPE, 15m CUT

5=NO. OF SLIP SURFACES 5=NO. OF RADII 2=NO. OF FUNCTIONS

SLIP NO.	X-COORD.	Y-COORD.	RADIUS	ITERATION NO.	LAMBDA	FACTOR OF SAFETY (MOMENT)	FACTOR OF SAFETY (FORCE)
1	66.250	190.000	11.950	1	.0000	14.475	15.065
1	66.250	190.000	11.950	8	.0000	14.475	14.104
2	66.250	190.000	18.273	1	.0000	3.896	3.965
2	66.250	190.000	18.273	20	.0000	3.896	999.000
3	66.250	190.000	24.590	1	.0000	2.471	2.489
3	66.250	190.000	24.590	13	.0000	2.471	999.000
4	66.250	190.000	30.840	1	.0000	1.908	1.927
4	66.250	190.000	30.840	11	.0000	1.908	999.000
5	66.250	190.000	37.000	1	.0000	1.605	1.634
5	66.250	190.000	37.000	14	.0000	1.605	999.000

! SUMMARY OF MINIMUM FACTORS OF SAFETY !

MOMENT EQUILIBRIUM: FELLENIUS OR ORDINARY METHOD
66.2500=X-COOR. 190.0000=Y-COOR. 37.0000=RADIUS 1.605=F.S.
MOMENT EQUILIBRIUM: BISHOP SIMPLIFIED METHOD
66.2500=X-COOR. 190.0000=Y-COOR. 18.2727=RADIUS 3.896=F.S.
FORCE EQUILIBRIUM: JANBU SIMPLIFIED METHOD (NO Fo FACTOR)
66.2500=X-COOR. 190.0000=Y-COOR. 11.9504=RADIUS 14.104=F.S.

NORMAL TERMINATION OF PC-SLOPE

APPENDIX C

REFERENCES

APPENDIX C

REFERENCES

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